

# FILE



**Washington State  
Department of Transportation**

Memorandum

March 1, 2004

TO: J. Kapur/M. Anderson  
Bridge and Structures, 47340  
*DP*

FROM: T.M. Allen/D.V. Jenkins  
EEP Geotechnical Division, 47365

SUBJECT: SR101, XL1640A  
Hoquiam River and Simpson Avenue Bridge 101/125W  
Maintenance Turnout  
Geotechnical Report

Attached with this memorandum is the *Geotechnical Report* for the design and construction of the proposed maintenance turnout for the Hoquiam River – Simpson Avenue Bridge 101/125W. The report addresses the following:

- Field investigation and testing
- Subsurface conditions and site seismicity
- Recommendations for drilled shaft foundations
- Construction considerations

If you have questions or require further information, please contact David Jenkins at (360) 709-5455 or Jim Cuthbertson at (360) 709-5452.

TMA  
DTM:dtm  
Enclosure

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## GEOTECHNICAL REPORT

# Hoquiam River – Simpson Bridge No. 101/125W – Maintenance Turnout

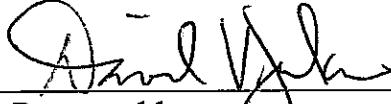
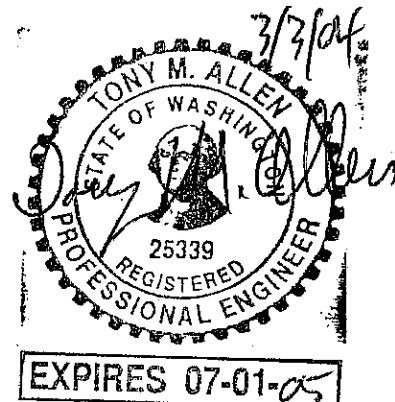
## Grays Harbor County, Washington

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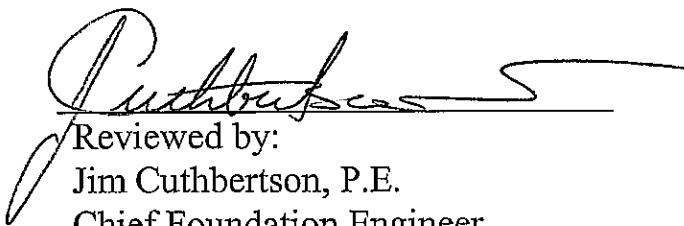
XL-1640A



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State Geotechnical Engineer



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Reviewed by:  
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March 1, 2004



**Washington State  
Department of  
Transportation**  
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## **1. INTRODUCTION**

### **1.1. GENERAL**

This report presents the results of our geotechnical investigation for the addition of the maintenance turnout of the SR-101 Hoquiam River – Simpson Avenue Bridge No. 101/125W. A vicinity map illustrating the project location is presented in Figure 1, Appendix A. Figure 2 presents a plan view showing the locations of all field test borings and Figure 3 provides a profile view showing the details of the subsurface conditions present at the site. This report provides geotechnical recommendations in LRFD format for foundation support of the new bridge. When the PS&E is completed for this project, our office will provide a *Summary of Geotechnical Conditions* for inclusion in the Special Provisions.

The analyses, conclusions, and recommendations provided in this report are based on the project description, and site conditions existing at the time of the field explorations. The exploratory borings are assumed to be representative of the subsurface conditions throughout the project area. If during construction, subsurface conditions differ from those described in the explorations, we should be advised immediately so that we may reevaluate our recommendations and provide assistance.

### **1.2. PROJECT DESCRIPTION**

The existing bridge consists of a 1978 ft long double bascule bridge constructed in 1927 under contract C1084. All piers of the bridge are supported on low capacity timber piles. The maintenance turnout bridge will be constructed south of the westerly approach between timber bents #36 and #39. The parking structure is supported on two columns. The single span structure is 46.5 ft long and approximately 13 ft wide. This new structure will be used to park vehicles during bridge inspection, maintenance and lift span operation. A sidewalk, cantilevered off the existing bridge, will be constructed between the parking structure and lift-span control house. A pre-cast concrete flat slab will support the roadway section of the parking structure. We understand the maintenance turnout will be structurally independent of the existing bridge. We are proposing that both piers of the maintenance turnout structure be supported on drilled shafts utilizing permanent steel casing during construction.

## **2. PROJECT SUBSURFACE CONDITIONS**

Subsurface conditions in the project area were explored by rotary drilling, standard penetrometer testing, cone penetrometer testing and a laboratory testing program. Appendix C provides a detailed discussion and all test hole data obtained in the field exploration program. Please note that the edited logs of the test borings should be made available to all prospective bidders, and included in the contract documents. Appendix D provides a discussion and all data obtained in the laboratory testing program.

## **2.1. REGIONAL GEOLOGY**

The geological units identified in the area are generally grouped based on geologic history and engineering characteristics. Quaternary alluvium is predominantly silt and/or sand with lesser and varying amounts of clay, organics, sea shells and gravel. The alluvium is typically very loose to loose in the upper 70 ft and increases in density with depth to loose to medium dense. In addition, the density of the material also increases with grain size (i.e. sand zones are typically more dense than silt zones).

At a depth of between 125 and 134 ft below the existing ground surface, the alluvium is underlain by Tertiary sedimentary bedrock. The sedimentary bedrock likely corresponds to the Montesano Formation mapped on the hills a few thousand feet north and east of the bridge site. Based on material recovered from the standard penetration test, we have interpreted the bedrock to consist of a conglomerate. A conglomerate is a weakly cemented sedimentary rock consisting of sand and gravels cemented with either silica and/or calcium carbonate. The Montesano Formation is described as middle to upper Miocene marine sedimentary rock consisting of coarse to fine grained sandstone, conglomerate, siltstone and mudstone.

## **2.2. SITE SURFACE CONDITIONS**

A site plan illustrating the locations of test holes and surface features is provided in Appendix A, Figure 2. The topography in the area is quite level with a very gentle slope breaking to the Hoquiam River. This area represents old tidal mud flats typical of land surrounding Grays Harbor. The elevation of the existing ground surface is approximately 12 ft above mean sea level.

## **2.3. SITE SUBSURFACE CONDITIONS**

The soil deposits encountered in the test borings at the Hoquaim River - Simpson Avenue Bridge have been grouped into soil units for geotechnical distinction. The soil units are grouped primarily on the basis of engineering properties and classification, and in general, reflect depositional environments as well. Subsurface profiles for the structure illustrating subsurface data and the interpreted conditions are provided in Appendix A, Figure 3.

Four soil units were identified during the field investigation. They are as follows:

**Unit 1** generally consists of very loose sandy silt with organics. Based on test boring TH-1-03, creosote treated piling and old concrete and brick foundations may be encountered in the upper portion of this unit. Unit 1 varies in thickness between 30 and 40 ft.

**Unit 2** consists of very loose to medium dense stratified layers of silty sand, poorly graded sand and sandy silt. All layers contain varying amounts of organics and sea shells. This layer averages 60 ft in thickness.

**Unit 3** consists of a very loose to loose sandy silt with organics. This layer varies in thickness between 24 and 30 ft.

**Unit 4** consists of a very dense conglomerate sedimentary rock. Conglomerate is composed of cemented gravels and sand. The contact elevation of this layer varies between -114 ft and -117 ft.

Logs of test boring and cone penetrometer data are contained in Appendix C. Laboratory test data is contained in Appendix D.

### **3. GROUND WATER**

Piezometers were not installed to monitor groundwater. Groundwater level observed during test drilling indicated water levels consistent with the water elevation in Hoquiam River. This area of the Hoquiam River is affected by tidal changes therefore the groundwater elevation is expected to vary daily with tide fluctuation.

## **4. SEISMOLOGICAL CONSIDERATIONS**

### **4.1. SITE SEISMICITY**

A convergence of the North American crustal plate and Juan de Fuca Plate is situated west of the project. The Juan de Fuca plate is subducting beneath the North American plate resulting in tectonic strain accumulation along the interface. The plate convergence can result in shallow earthquakes along the plate interface (thrust events), deep earthquakes within the subducted Juan de Fuca plate (intraplate normal-faulting events), and shallow earthquakes within the North American crust.

### **4.2. DESIGN EARTHQUAKE PARAMETERS**

For Seismic Design, an acceleration coefficient of 0.3g is recommended for this structure in accordance with the WSDOT Bridge Design Manual.

Design response spectra presented in the AASHTO guide specifications for seismic design of highway bridges are considered appropriate for seismic design. A type III soil profile response spectrum, with a site coefficient of 1.5 is recommended for seismic design. These recommendations are based on our review of a report titled "WSDOT Special Bridges Seismic Evaluation, Aberdeen Washington" dated June 1997 prepared by Shannon and Wilson Inc.

### **4.3. LIQUEFACTION POTENTIAL**

Liquefaction of saturated sands occurs when the sands are subject to cyclic loading. The cyclic loading causes the water pressure to increase in the sand reducing the intergranular stresses. As the intergranular stresses are reduced, the shearing resistance of the sand decreases. If pore pressures develop to the point where the effective stresses acting between the grains become zero, the soil will behave like a viscous fluid. Under this condition soil flow is possible. The effect of liquefaction can range from reduced shear strength to viscous fluid behavior.

The liquefaction potential of saturated soils is evaluated mainly on soil gradation, relative density, and the depth of the deposit, i.e., the vertical effective overburden stress. The potential for liquefaction is highest for loose, fine to medium grained, sandy and silty soils. Increasing fines content, i.e., silt and clay, decreases the potential for liquefaction. If a deposit has greater than 35% fines it is usually considered to be non-liquefiable. Due to their high hydraulic conductivity, gravel soils are less susceptible to liquefaction; however, they can liquefy depending on their fines content, thickness, areal extent and/or the drainage conditions at their boundaries. The potential for liquefaction of all cohesionless, granular soils decreases with increasing depth and relative density.

At the site of Hoquiam River-Simpson Bridge, the upper portion of unit 2 has many of the characteristics of liquefiable soils, i.e., it is loose, saturated and has relatively low fines content. The fines content of the unit varies between 7 and 61 percent. Ten of the 15 tested samples from unit 2 had fines contents from 7 to 30 percent.

Liquefaction potential of unit 2 was based on SPT results and was evaluated using the simplified procedure proposed by Seed and Idriss, 1982. An acceleration of 0.30g was assumed. The liquefaction analysis indicated that within unit 2 the factor of safety against liquefaction is generally less than 1.0. Current state-of-practice is to assume that liquefaction may occur at factors of safety less than 1.1. The liquefiable zone used in the subsequent analyses, e.g., calculation of downdrag loads, was assumed to lie between the elevations of 5 and -55 feet. The upper soil Unit 1 is assumed to contribute to downdrag forces along with approximately the upper 30 ft of Unit 2.

#### **4.4. LIQUEFACTION INDUCED LATERAL SPREADING AND STRAIN**

Although no embankments are planned for construction of the maintenance turnout the risk of lateral spreading from the sloping ground between the structure and river is moderate. However, designing this structure to withstand lateral spreading is cost prohibitive.

### **5. GEOTECHNICAL RECOMMENDATIONS**

#### **5.1. DRILLED SHAFT FOUNDATION RECOMMENDATIONS PIER 1 AND 2**

A drilled shaft foundation is recommended for support of both piers. Drilled shafts transfer the load to deeper, more competent strata thus they will support the bridge if the upper soils settle due to the liquefaction of unit 2. Piles are not recommended due to the potential for pile driving induced liquefaction that could result in settlement of the existing pile supported bridge.

Recommendations for drilled shaft design are given below. Enclosed in Appendix A, Figure 4, shows the ultimate capacity for service, strength and extreme event limit states for Pier 1 and 2, for 6 ft diameter shafts. The figures show the net load that can be applied at the top of the shaft.

*The weight of the shaft has not been deducted from the compressive capacity in the figures and is not included in the uplift capacity.*

The capacity figures for shafts at Piers 1 and 2 are for 6 ft diameter. Separate plots for ultimate skin friction ( $Q_s$ ) and ultimate end bearing ( $Q_p$ ) are provided on the figures. At a given depth on the figures, the factored resistance ( $Q'$ ) can be determined by adding the ultimate skin friction multiplied by its resistance factor ( $\phi_s$ ) and the ultimate end bearing multiplied by its resistance factor ( $\phi_p$ ) as shown in the following equation:

$$Q' = Q_s \cdot \phi_s + Q_p \cdot \phi_p$$

For the service limit state, the settlement of the shaft foundations will be less than 1 inch provided the shafts are at or below the minimum tip elevation of -126 ft. Settlement will occur as the loads are applied. Post construction settlement should be negligible. We are recommending a minimum 12 ft embedment into Unit 4 (2-shaft diameters). Based on this recommendation, a minimum tip elevation for Piers 1 and 2 has been established at -126 ft. A deeper penetration into Unit 4 may be required for lateral stability.

For axial load reduction of shaft groups we recommend a group reduction factor of 1 be used for groups with center-to-center spacing of 3b or greater. A factor of 1 is recommended since 3 or fewer shafts are expected in each shaft group.

## 5.2. RESISTANCE FACTORS FOR SHAFT DESIGN

We recommend that the resistance factors shown in Table 1 be used when evaluating the different limit states.

Table 1

Limit State	Resistance Factor $\phi$		
	Skin Friction $Q_s$	End Bearing $Q_b$	Uplift
Strength	0.65	0.5	0.55
Service	1.00	1.00	N/A
Extreme	1.00	1.00	0.75

## 5.3. LATERAL LOAD ANALYSIS (PIERS 1, AND 2)

We have evaluated the site conditions with regard to the *Design Manual for Foundation Stiffnesses Under Seismic Loading*. Based on our review, soil conditions at Piers 1 and 2 cannot be readily correlated to any of the standard soil profiles.

For this reason P-Y curve data for lateral load analysis of the drilled shafts Piers 1 and 2 are provided in Appendix B.

We recommend that the group reduction factors shown in Table 2 be used for lateral analysis.

Table 2

Pile or Shaft Spacing	Reduction factor for multiple row groups, or in direction parallel to single row	Reduction factor for single row groups for loading perpendicular to the row.
6D	0.9	1.0
5D	0.8	1.0
4D	0.65	0.9
3D	0.5	0.8
2D	0.4	0.6

#### 5.4. DOWNDRAg LOADS AND SIDE FRICTION LOSSES

Downdrag loads are created when the soil moves downward relative to the shaft, thus transferring load onto the shaft. Excessive downdrag loads can result in either structural failure of the shaft or bearing capacity failure of the bearing layer. The usual mechanisms that generate downdrag loads are post-construction settlements due to the placement of an embankment and/or settlements induced by the liquefaction of one or more soil layers.

Downdrag loads caused by post-construction settlements most often occur in shafts that pass through a soft, compressible soil and then bear in a stiffer layer. As the new embankment fill initiates consolidation of the softer material, the side friction forces in the fill are reversed and begin acting downward on the shaft. Hence, settlement induced downdrag loads result in the loss of the side friction capacity of the shaft in the consolidating units.

When liquefaction occurs, it results in a sudden settlement of the liquefied layer. As this layer settles, i.e., as the excess pore pressure dissipates, it creates downdrag loads on the shaft which must be carried by the lower, non-liquefied soils. Soils overlying the liquefied layer also will generate downdrag loads on the shaft as they settle in response to the liquefaction of the underlying layer. Liquefaction also results in the loss of the side friction capacity of the shaft in the liquefied zone as well as in soils overlying it.

Estimated downdrag loads and side friction losses due to liquefaction induced settlements were estimated at 300 tons for 6 ft diameter shafts. The downdrag loads and side friction losses are numerically equivalent.

Downdrag loads should be added directly to the factored bridge loads when evaluating the shaft capacity required for the various limit states. Side friction losses should be subtracted from the unfactored side capacity curves (Figure 4).

## **6. CONSTRUCTION CONSIDERATIONS**

It may be possible to drill the shafts full depth using polymer slurries, but due to the extreme looseness of the native soils, we recommend the use of permanent casing to prevent ground caving. Vibrating the casing is not recommended because of the potential of liquefaction and the effects that it may have on the existing bridge structure. Liquefaction induced settlement could result in several inches of deflection of the existing bridge. Therefore we recommend the contract specify casing advancement using a casing-oscillating or rotating method. The contract should be written to allow the use of temporary casing provided the contractor has a drill rig capable of removing the casing during shaft construction. The estimated bottom elevation of the permanent casing is -117 ft at both piers. The bottom elevation of the casing was established to insure the casing is seated several feet into the Unit 4 sedimentary rock. Slurry will be required during removal of the spoils.

## **APPENDIX A - FIGURES**

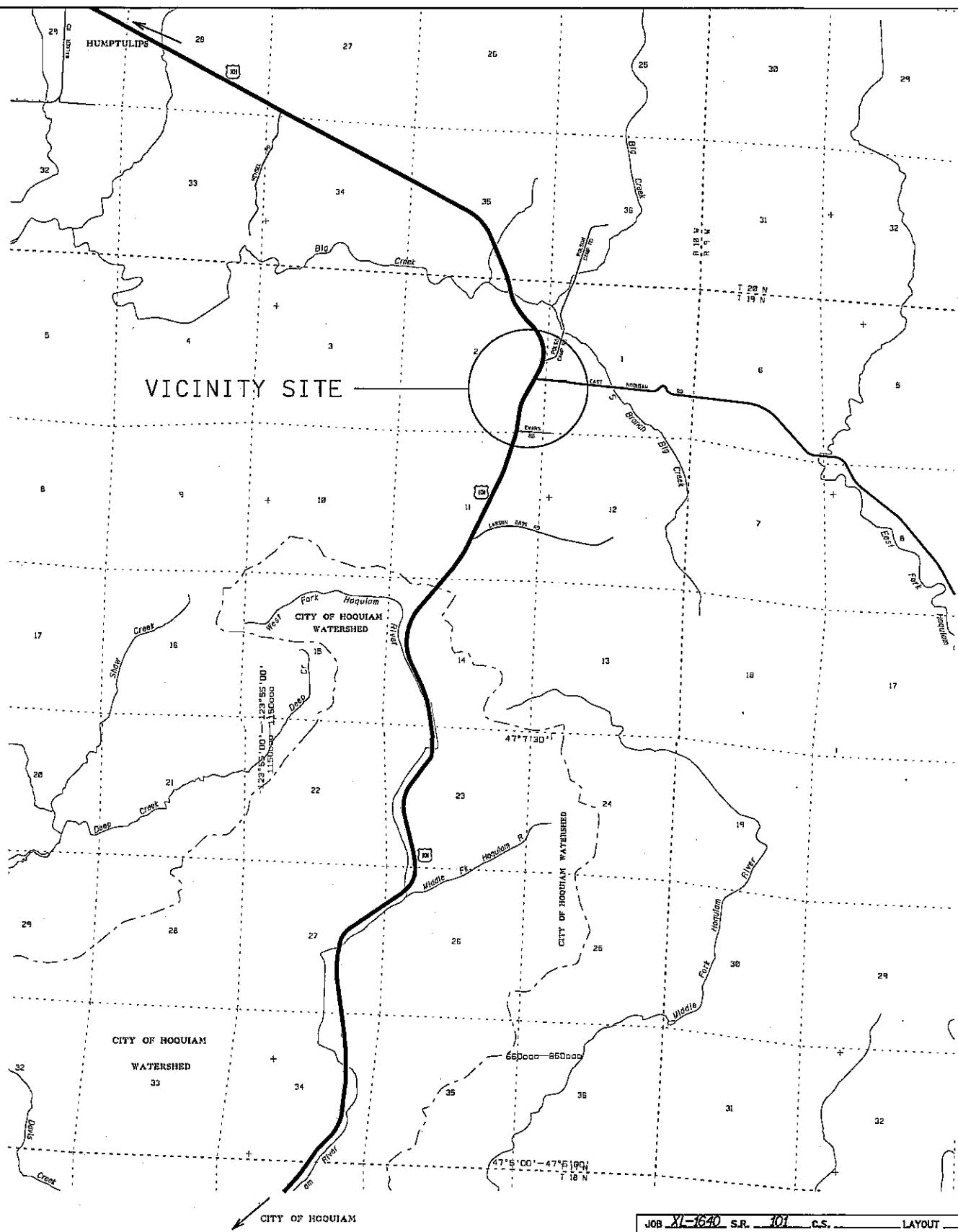
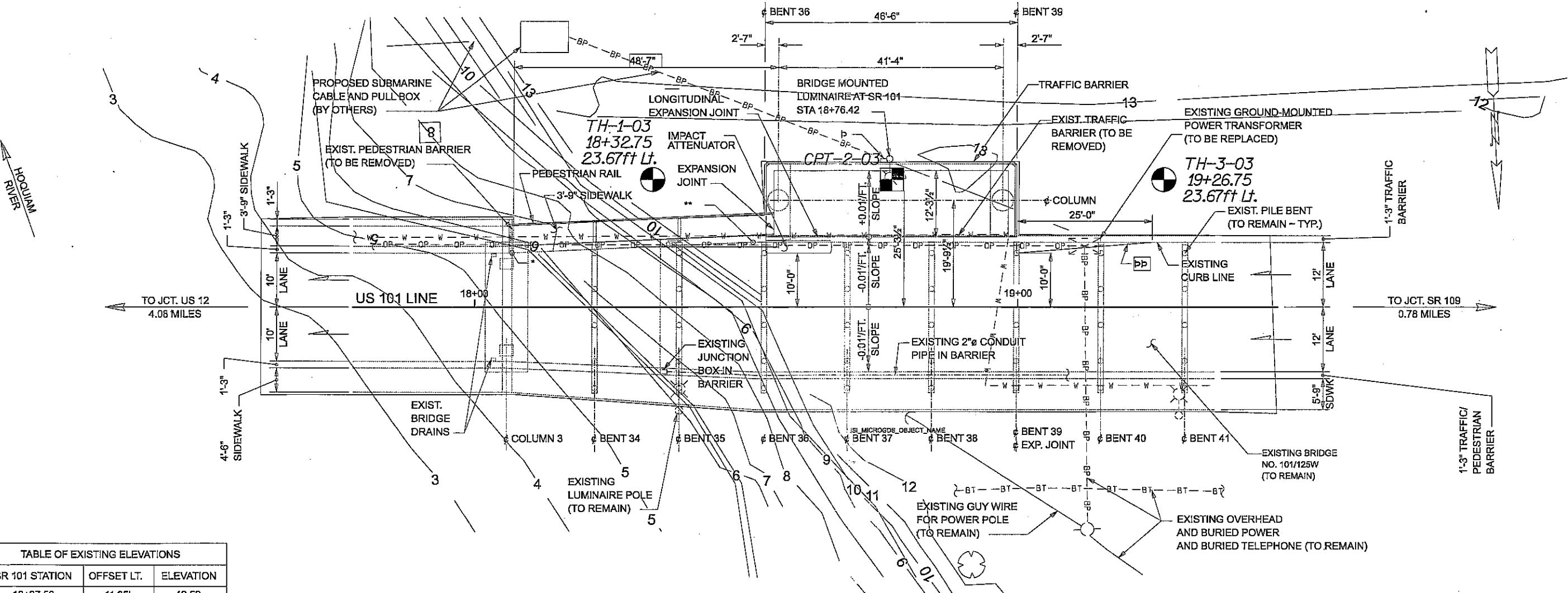


FIGURE 1: SITE MAP

JOB # XL-1640 S.R. 101 C.S. LAYOUT		
West Hoquiam Bridge Maintenance Turnout		
WASHINGTON STATE TRANSPORTATION COMMISSION DEPARTMENT OF TRANSPORTATION		DATE 2/2004
MATERIALS BRANCH T. E. BAKER MATERIALS ENGINEER		SCALE N.T.S. VERT. HORIZ.
		SHEET ____ OF ____
		DRAWN BY W.M.

SEC. 12, T.17N., R.10W., W.M.  
CITY OF HOQUIAM



*Test Boring*



*Cone Penetrometer*

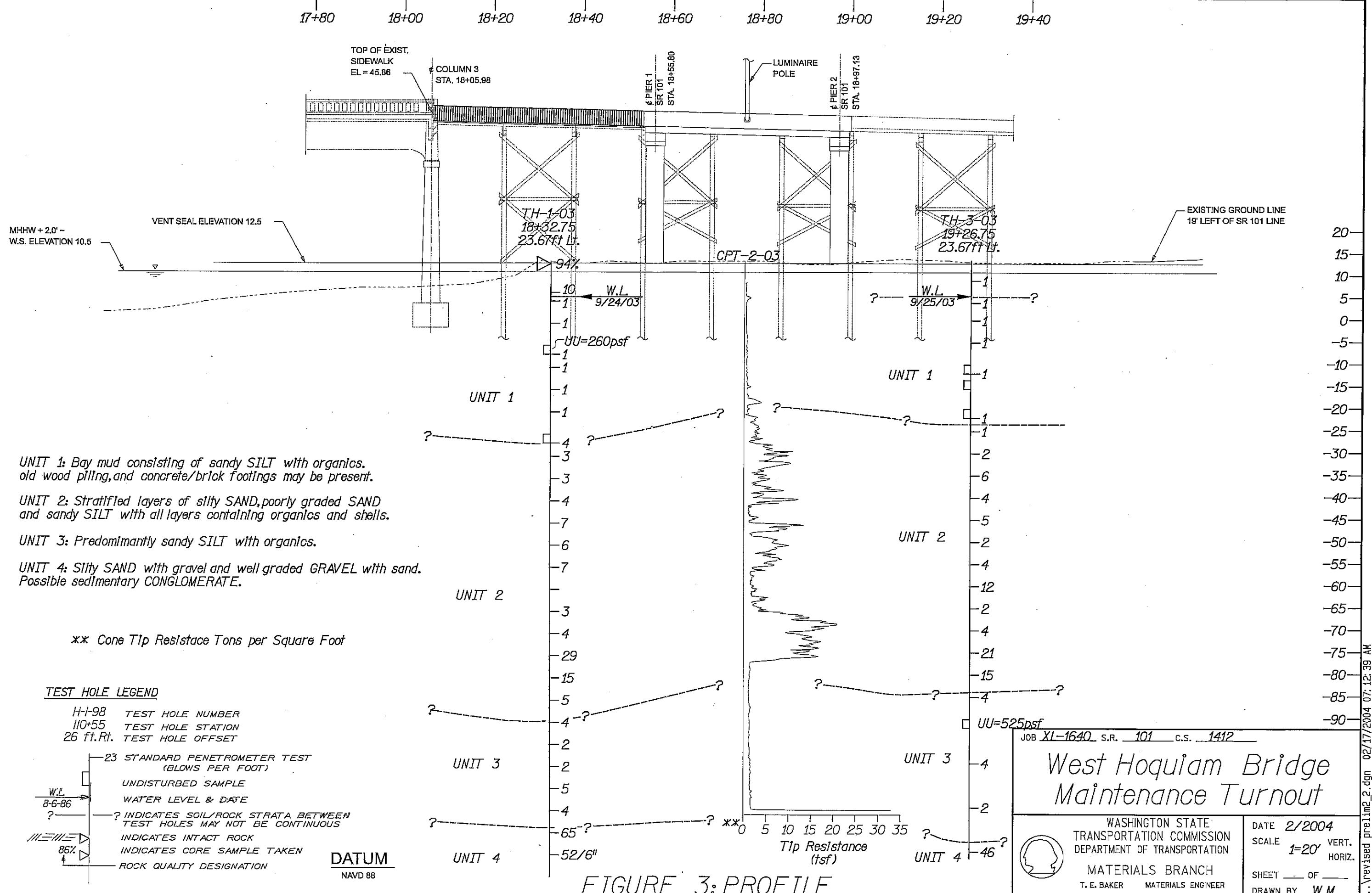
FIGURE 2: PLAN

JOB XL-1640 S.R. 101 C.S. 1412

*West Hoquiam Bridge  
Maintenance Turnout*

WASHINGTON STATE  
TRANSPORTATION COMMISSION  
DEPARTMENT OF TRANSPORTATION  
MATERIALS BRANCH  
T. E. BAKER MATERIALS ENGINEER

DATE 2/2004  
SCALE 1=20' VERT.  
HORIZ.  
SHEET        OF         
DRAWN BY W.M.



*FIGURE 3: PROFILE*

**SR-101 Hoquiam River-Simpson Bridge 101/125W**

Pier(s)  
1 & 2  
Diameter  
6.0 ft  
Casing  
None

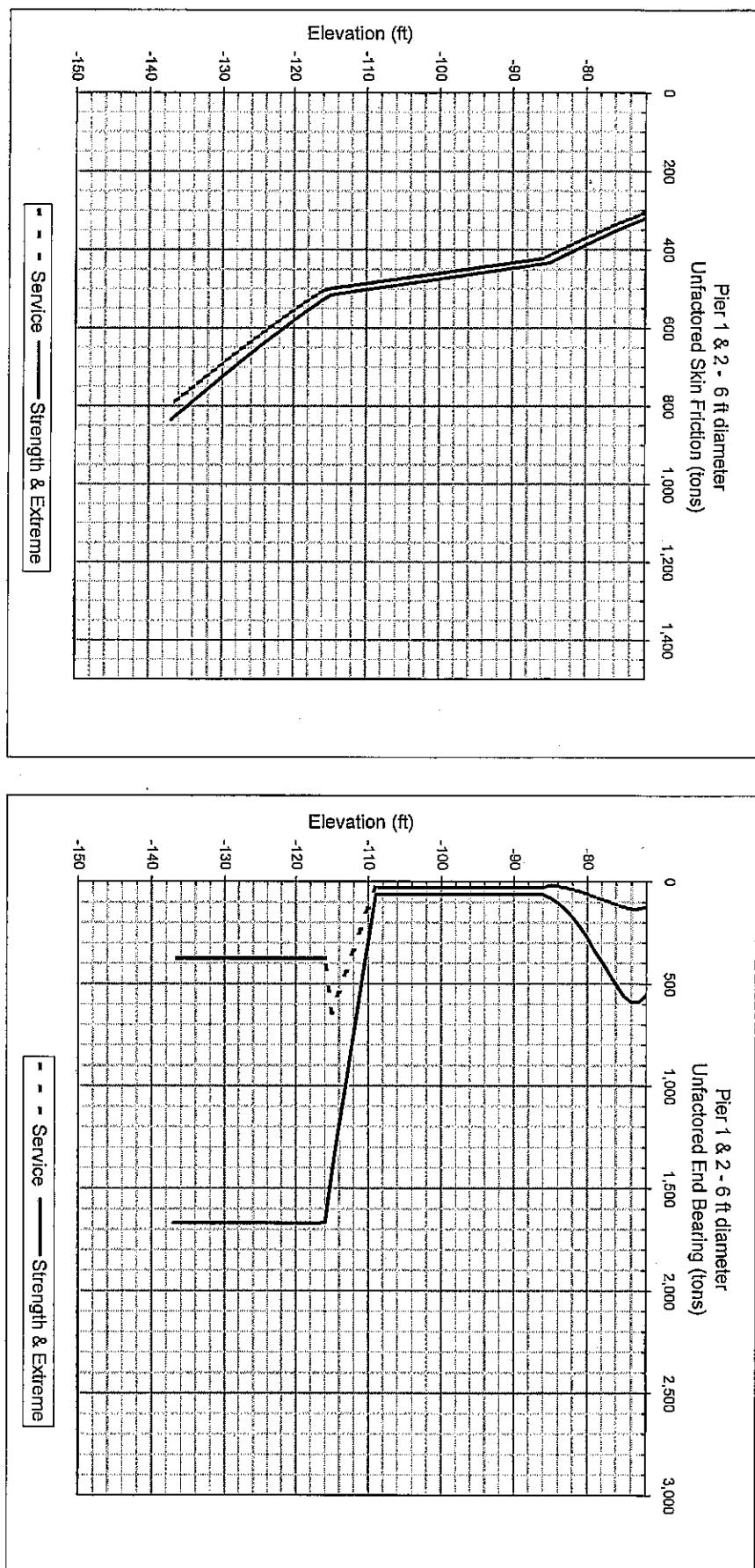


Figure 4: Axial Capacity for 6 ft Diameter Drilled Shafts for Piers 1 and 2

## **APPENDIX B – P/Y DATA**

SR101

**101/125W Hoquiam River/Simpson Avenue Bridge  
Maintenance Turnout  
P-Y CURVE SOIL DATA  
for LPILE Program**

Applies to Pier: 1 & 2

Ground Surface Elevation (ft): 12.5

Estimated top of Shaft Elevation (ft): 5.0

DYNAMIC ANALYSIS

## **APPENDIX C – FIELD EXPLORATION**

## **INTRODUCTION**

The field exploration program for the project consisted of drilling 2 test borings, performing Standard Penetration Tests (SPT), and discretely sampling soil horizons. In addition 1 cone penetrometer test was performed. The information obtained during the field exploration was used in conjunction with existing information obtained from the previous studies and final construction records of the existing Hoquiam River Bridge were used to characterize the subsurface conditions throughout the project area. The edited logs of the field test are attached. The edited logs of the test borings and the results of the cone penetration test should be included in the contract documents.

## **TEST BORINGS**

Standard Penetration Tests (SPT), in general, were performed at five-foot intervals in the test borings. Portable penetrometer tests were conducted every 1.5 feet. Disturbed soil samples from the SPT, and hand holes were visually classified in the field then submitted to the E&EP Materials Laboratory for more detailed classification and testing.

## **CONE PENETROMETER TESTING**

Cone Penetration Testing uses a cylindrical cone, pushed vertically into the ground at a constant rate of penetration of 20mm/s. During penetration, measurements are made of the cone resistance, the side friction against the cylindrical shaft and, in piezocone tests, the pore water pressure generated at penetration by the cone.

The measurements are made and recorded using electrical devices, the frequency of the readings provides a detailed picture of the variation of the measured parameters with penetration depth. Cone diameters are 10 or 15 cm<sup>2</sup>. Depths up to 100 metres can be reached.

**TEST BORING LOGS**



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No. XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Project West Hoquiam Bridge - Maintenance-Turnout

Sheet 1 of 6

Site Address Vicinity Simpson Ave. and Hoquiam River Bridge

Driller James Fetterly

Lic# 2507

Start September 22, 2003 Completion September 24, 2003 Well ID#  Equipment CME 850 w/ autohammer

Station 18+32.75 Offset 13.67' LT. Casing (HWT 4" x 12.0')(HQ 3" 142.0") Method Wet Rotary

Northing \_\_\_\_\_ Easting \_\_\_\_\_ Latitude \_\_\_\_\_ Longitude \_\_\_\_\_

County Grays Harbor Subsection NW 1/4 of the SW 1/4 Section 12 Range 10 WWM Township 17 N

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material		Groundwater	Instrument
			10	20	30	40								
1							RQD 94 FF 2	C-1			Top surface cobbles, boulders, rip-rap fill material, abandoned concrete pilings made with brick and cement with rebar steel. 0.0' to 0.5' overburden. (Changed at 7.2'). CONCRETE, coarse grained, slightly weathered, moderately strong rock, no HCl reaction. Discontinuities are closely spaced and in good condition, Percent. Recovered 97.0%			
2							3 7 3 (10)	D-2			Well graded GRAVEL with sand, with creosote wood, surrounded, medium dense, brown, moist, Stratified, no HCl reaction, (Note drill and sample through creosote log from 7.2' to 8.3'. Changed at 8.5' as indicated by drilling and wash return. 100% drilling fluid return.			
3							1 1/12 (1)	D-3	GS MC		Length Recovered 0.8 ft, Length Retained 0.8 ft 09/24/2003			
4							1/18 (1)	D-4	GS MC		ML, MC=80% Sandy SILT with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (100% drilling fluid loss). Length Recovered 1.5 ft, Length Retained 1.0 ft			
5											ML, MC=73% Sandy SILT, very loose, dark gray, moist, Stratified, no HCl reaction, (Note advanced hwt casing to 10.5' started getting 100% drilling fluid back). Length Recovered 1.5 ft, Length Retained 1.0 ft			
6		x x						U-5 A	GS MC		MH, MC=68%, PI=16 Elastic SILT, very loose, dark gray, moist, Stratified, no			



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Sheet 2 of 6

Driller James Fetterly Lic# 2507

Project West Hoquiam Bridge - Maintenance-Turnout

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40							
6.5	x x x					1/18 (1)	B C D E F	AL UU GS MC		HCl reaction, (Note sample sank with weight of rods and hammer). Length Recovered 2.0 ft, Length Retained 2.0 ft ML, MC=74% Sandy SILT, very loose, dark gray, moist, Laminated, no HCl reaction, traces of organic and seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
7.5						1/18 (1)	D-7	GS MC		ML, MC=71% Sandy SILT, very loose, dark gray, moist, Laminated, no HCl reaction, traces seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
8.5						1/18 (1)	D-8			Sandy SILT with organics lenses, very loose, dark gray, moist, Laminated, no HCl reaction, traces of seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
9.5						1/18 (1)	D-9	GS MC		ML, MC=60% Sandy SILT with organics, very soft, dark gray, moist, Laminated, no HCl reaction, (Note very little drilling fluid loss starting at 29.0'). Length Recovered 1.5 ft, Length Retained 1.0 ft		
10.5						1/18 (1)	U-10 A B C D			SILT, with sand, organic, wood twigs, very loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.7 ft, Length Retained 1.7 ft		
11.5						1 2 2 (4)	D-11	GS MC		SM, MC=54% Silty SAND, very loose, dark gray, moist, Stratified, no HCl reaction, traces of wood particles Length Recovered 1.5 ft, Length Retained 1.0 ft		
12.5						1 2	D-12	GS MC		SM, MC=53% Silty SAND with organics, wood debris, very loose, dark		
13.5												
45												



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Sheet 3 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Lic# 2507

Depth (ft)  Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40							
14						1 (3)	▼			gray, moist, Stratified, no HCl reaction, traces of seashell. (Note sulfur like smell). Length Recovered 1.5 ft, Length Retained 1.0 ft		
15						1 (3)	▼	D-13		Silty SAND, very loose, dark gray, moist, Stratified, no HCl reaction, traces of organic, wood particles and seashell. Length Recovered 1.0 ft, Length Retained 1.0 ft		
16						1 (4)	▼	D-14				
55						1 (4)	▼	D-14		Silty SAND, organics, wood debri and seashell, loose, dark gray, moist, Stratified, no HCl reaction, laminated with fine grained sand and small surrounded gravel. (Note sulfur like odor). Length Recovered 1.5 ft, Length Retained 1.0 ft		
17						2 (7)	▼	D-15	GS MC	SP-SM, MC=29% Poorly graded SAND with silt, loose, dark gray, moist, Stratified, no HCl reaction, laminated with organic, traces of seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
18						2 (7)	▼	D-15	GS MC	SP-SM, MC=29% Poorly graded SAND with silt, loose, dark gray, moist, Stratified, no HCl reaction, laminated with organic, traces of seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
19						1 (6)	▼	D-16		Silty SAND, loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
20						1 (6)	▼	D-16		Silty SAND, loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
21						1 (3)	▼	D-17		Silty SAND, organics and wood particles, loose, dark gray, moist, Stratified, no HCl reaction, traces mica sand		
70												



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No. XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Sheet 4 of 6

Driller James Fetterly Lic# 2507

Project West Hoquiam Bridge - Maintenance-Turnout

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40							
22						4 (7)	▼			and seashell. Length Recovered 1.5 ft, Length Retained 1.0 ft		
23						1 5 7 (12)	▼	D-18		Silty SAND, organics, mica sand and wood particles, medium dense, dark gray, moist, Stratified, no HCl reaction, very little drilling fluid loss. Length Recovered 1.5 ft, Length Retained 1.0 ft		
24						1 2 1 (3)	▼	D-19		Silty SAND, organic lenses, very loose, dark gray, moist, Laminated, no HCl reaction, traces of wood particles and mica sand. Length Recovered 1.5 ft, Length Retained 1.0 ft		
25						1 2 2 (4)	▼	D-20		Silty SAND, fine grained sand lenses, very loose, dark gray, moist, Laminated, no HCl reaction, traces of organic and mica sand. Length Recovered 1.0 ft, Length Retained 1.0 ft		
26						10 13 14 (29)	▼	D-21	GS MC	SP-SM, MC=43% Poorly graded SAND with silt and wood debri, dense, dark gray, wet, Stratified, no HCl reaction Length Recovered 1.3 ft, Length Retained 1.0 ft		
27						5 6	▼	D-22	GS MC	SP-SM, MC=28% Poorly graded SAND with silt and organics and wood		
28												
29												



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No. XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Sheet 5 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Lic# 2507

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material		Groundwater	Instrument
		10	20	30	40								
29						9 (15)	▼			debris, medium dense, dark gray, moist, Laminated, no HCl reaction, traces of organic and wood particles Length Recovered 1.5 ft, Length Retained 1.0 ft			
30													
100						1 (5)	▼	D-23	GS MC	SM, MC=65% Silty SAND, medium stiff, dark gray, moist, Laminated, no HCl reaction, traces of peat. Length Recovered 1.5 ft, Length Retained 1.0 ft			
31													
105						1 (4)	▼	D-24	GS MC	ML, MC=51% Sandy SILT, very loose, dark gray, moist, Homogeneous, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft			
32													
33						1 (2)	▼	D-25		Sandy SILT, very loose, dark gray, moist, Homogeneous, no HCl reaction, traces of organic Length Recovered 1.5 ft, Length Retained 1.0 ft			
110													
34													
115						1/6 (2)	▼	D-26	GS MC	ML, MC=58% Sandy SILT, very loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft			
35													
36						1 (2)	▼	D-27	GS MC	ML, MC=59% Sandy SILT, medium stiff, dark gray, moist, Laminated,			
120													



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 12.9 ft (3.9 m)

HOLE No. TH-1-03

Sheet 6 of 6

Driller James Fetterly Lic# 2507

Project West Hoquiam Bridge - Maintenance-Turnout

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40							
37						3 (5)	▼			no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
38						1 (4)	▼	D-28		Sandy SILT, organics, very loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
39						22 (45)	▼	D-29	GS MC	SM, MC=33% Silty SAND with gravel, very dense, dark gray, moist, Stratified, no HCl reaction, some wood debris, gravel are well graded and subrounded. Length Recovered 1.5 ft, Length Retained 1.0 ft		
40						33 52/6 (52)	▼	D-30	GS MC	GW, MC=6% Well graded GRAVEL with sand, subrounded, very dense, dark gray, moist, Homogeneous, no HCl reaction Length Recovered 0.5 ft, Length Retained 0.5 ft		
41										End of test hole boring at 139 ft below ground elevation.		
42										This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
43										(Note drilled to depth 139.0' screws came lose from advancer had to pull hq casing. (No sample taken at 139.0' silty Gravel wit sand till material as indicated by drilling. Ended hole at depth 139.0').		
44												
145												



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

HOLE No. TH-3-03

Sheet 1 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly Lic# 2507

Site Address Vicinity Simpson Ave. and Hoquiam River Bridge

Inspector Cleo Andrews

Start September 24, 2003 Completion September 25, 2003 Well ID# \_\_\_\_\_ Equipment CME 850 w/ autohammer

Station 19+26.75 Offset 23.67' LT. Casing (HWT 4" x 12.0')(HQ 3" x 1140.0') Method Wet Rotary

Northing \_\_\_\_\_ Easting \_\_\_\_\_ Latitude \_\_\_\_\_ Longitude \_\_\_\_\_

County Grays Harbor Subsection NW 1/4 of the SW 1/4 Section 12 Range 10 WWM Township 17 N

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40						
1									Top surface crushed gravel, cobbles, boulders, straw. 0.0' to 4.5' rip-rap cobbles, boulders, sawdust fill material. 1005 drilling fluid return.		
5						1/18 (1)	D-1		ORGANIC SOIL, very soft, dark brown, moist, Homogeneous, no HCl reaction, (Sawdust), fill material. Length Recovered 1.0 ft, Length Retained 1.0 ft		
10						1/18 (1)	D-2	GS MC	ML, MC=71% SILT with sand and organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Note took moisten can MC-2a from same depth). Changed at 8.0' as indicated by drilling and wash return. Length Recovered 1.0 ft, Length Retained 1.0 ft	▽	09/25/2003
15						1/18 (1)	D-3		SILT with sand with organics and seashell, very loose, dark gray, moist, Stratified, no HCl reaction, some pieces of wood fragement. (Took moisten can MC-3a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		
20						1/18 (1)	D-4		SILT with sand with organics and seashell, very loose, dark gray, moist, Stratified, no HCl reaction, (Took moisten can sample MC-4a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		



Washington State  
Department of Transportation

## LOG OF TEST BORING

Start Card S 23816

Job No. XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

HOI E No TH-3-03

Sheet 2 of 6

Sheet 2 of 6

Lic# 2507

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material			Groundwater	Instrument
			10	20	30	40									
7									U-5		No Recovery				
25							1/18 (1)	D-6			SILT with sand with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Took moister can sample MC-6a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft				
30								U-7 A to F			SILT with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Note sample sank with weight of rods and hammer 0 psi). Length Recovered 2.0 ft, Length Retained 2.0 ft				
35								U-8 A to F			SILT, very loose dark gray, moist, laminated with organic and sand lenses. Length Recovered 2.0 ft, Length Retained 2.0 ft				
40							1/18 (1)	D-10			SILT with sand with organics, very loose, dark gray, moist, Stratified, no HCl reaction, laminated with fine grained sand lenses. (Took moister can sample MC-10a from same depth). 100% drilling fluid return. Length Recovered 1.5 ft, Length Retained 1.0 ft				
45							1/18 (1)	D-11	GS MC		SM, MC=58% Silty SAND with organics, very soft, dark gray, moist, Stratified, no HCl reaction, (Took moister can sample MC-11a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft				
47							1/6 1 1 (2)	D-12	GS MC		SM, MC=56% Silty SAND, very loose, dark gray, moist, Homogeneous, no HCl reaction, traces of organic and mica sand. Very little drilling fluid loss in upper rip-rap zone. (Note				



Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

HOLE No. TH-3-03

Sheet 3 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Lic# 2507

Depth (ft) Meters (m)	Profile	Standard Penetration Blows/ft				SPT Blows/6' (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
		10	20	30	40							
14										changed to sand as indicated by drilling and wash return at 46.0'). Length Recovered 1.0 ft, Length Retained 1.0 ft		
15										SP-SM, MC=39% Poorly graded SAND with silt and organics and wood chunks, loose, grayish black, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
16										SM, MC=45% Silty SAND with organics and shells, very loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
17										SM, MC=37% Silty SAND with organics, very loose, dark gray, wet, Stratified, no HCl reaction. Length Recovered 1.0 ft, Length Retained 1.0 ft		
18										ML, MC=58% Sandy SILT, very loose, dark gray, moist, Stratified, no HCl reaction, sand is fine grained layers are 1" and horizontal. (Took moister can sample MC-16a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		
19										Silty SAND with organics, seashell and decayed wood debris, very loose, dark gray, moist, Stratified, no HCl reaction Length Recovered 1.0 ft, Length Retained 1.0 ft		
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Washington State  
Department of Transportation

# LOG OF TEST BORING

Start Card S 23816

HOLE No. TH-3-03

Sheet 4 of 6

Job No. XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Lic# 2507

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab	Tests	Description of Material	Groundwater	Instrument
22			10 20 30 40	5 6 6 (12)	D-18	GS MC			SM, MC=31% Silty SAND, medium dense, dark gray, moist, Laminated, no HCl reaction, traces of organic. Length Recovered 1.5 ft, Length Retained 1.0 ft		
23				33 2 2 (2)	D-19	GS MC			ML, MC=56% Sandy SILT with organics, sand, lenses, very loose, dark gray, moist, Laminated, no HCl reaction, some mica sand lenses. (Took moister can sample MC-19a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		
24				2 2 2 (4)	D-20	GS MC			SM, MC=42% Silty SAND, very loose, dark gray, moist, Laminated, no HCl reaction, traces of wood wood particles. Length Recovered 1.5 ft, Length Retained 1.0 ft		
25				6 11 10 (21)	D-21	GS MC			SM, MC=34% Silty SAND with organics, mica sand lenses, medium dense, dark gray, moist, Laminated, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
26				8 9 6 (15)	D-22	GS MC			SP-SM, MC=25% Poorly graded SAND with silt, medium dense, dark gray, moist, Laminated, no HCl reaction Length Recovered 1.5 ft, Length Retained 1.0 ft		
27											
28											
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Washington State  
Department of Transportation

## LOG OF TEST BORING

Start Card S 23816

Job No XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

HOLE No. TH-3-03

Sheet 5 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

Lic# 2507

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
-29			10 20 30 40							
-30				2 2 2 (4)	D-23			SILT with sand with organics, wood particles, very loose, dark gray, moist, Stratified, no HCl reaction, wood particles are decayed. (Took moister can sample MC-23a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		
100										
-31										
105										
-32					U-24	A to F	GS MC AL UU	CH, MC=63%, PI=304 Fat CLAY with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Took 150 psi to push undisturbed sample 2.0'). Length Recovered 2.0 ft, Length Retained 2.0 ft		
110										
-33										
115										
-34										
-35				1 1 3 (4)	D-25			SILT with sand with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Took moister can sample MC-25a from same depth). Length Recovered 1.5 ft, Length Retained 1.0 ft		
120										



Washington State  
Department of Transportation

## LOG OF TEST BORING

Start Card S 23816

Job No. XL-1640

SR 101

Elevation 13.1 ft (4.0 m)

HOLE No. TH-3-03

Sheet 6 of 6

Project West Hoquiam Bridge - Maintenance-Turnout

Driller James Fetterly

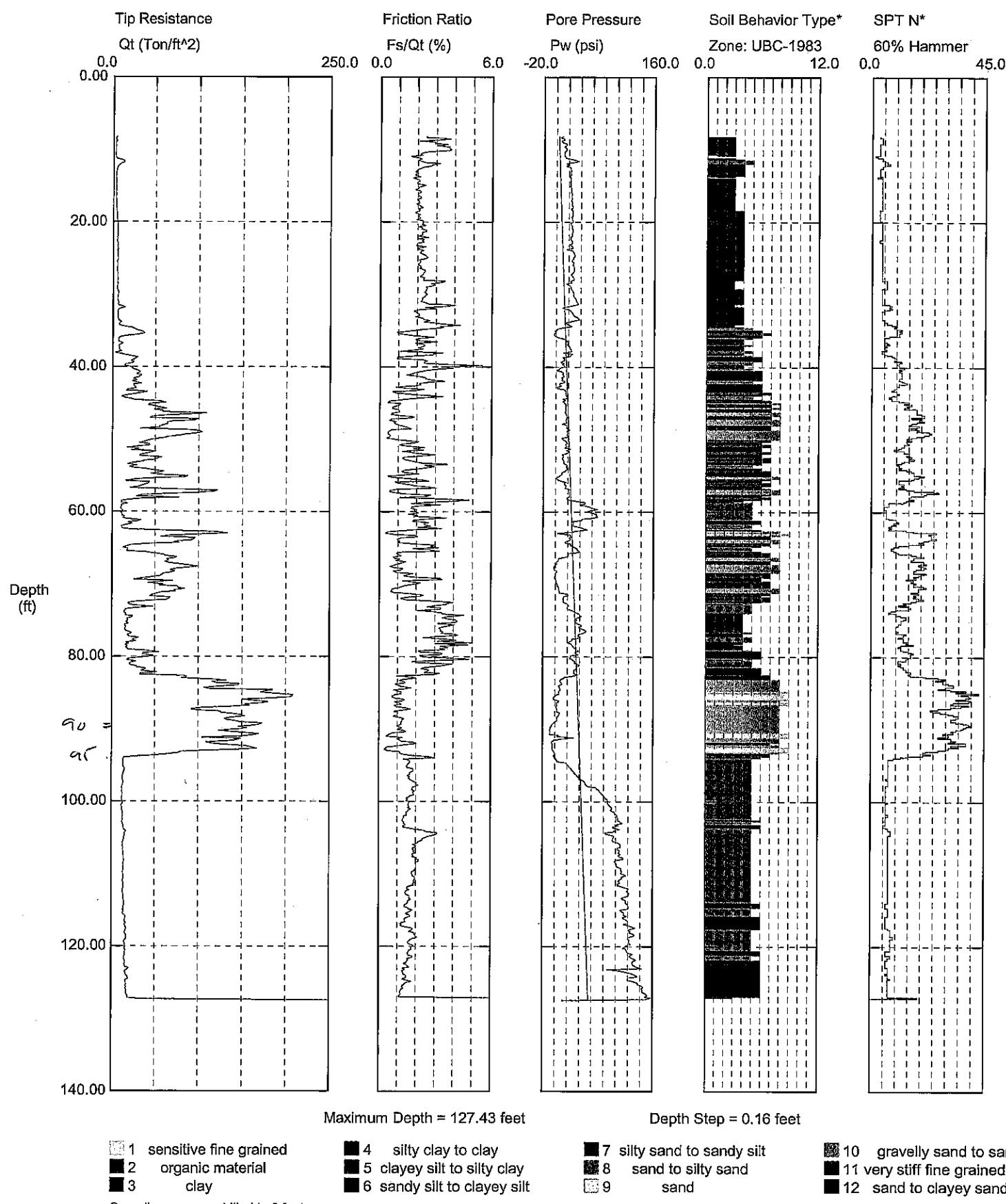
Lic# 2507

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab	Tests	Description of Material	Groundwater	Instrument
37			10 20 30 40								
38				1 1 1 (2)	D-26				SILT with sand with organics, very loose, dark gray, moist, Stratified, no HCl reaction, (Took moister can sample MC-26a from same depth). Note encountered some gravel at 132.0' as indicated by drilling. Length Recovered 1.5 ft, Length Retained 1.0 ft		
39											
40											
41		00° 00' C		9 20 26 (46)	D-27	GS MC			GW, MC=8% Well graded GRAVEL with sand, subrounded, dense, dark gray, moist, Homogeneous, no HCl reaction, (TIII material). Length Recovered 0.8 ft, Length Retained 0.8 ft		
42									Ended and abandoned hole using a bentonite cement slurry.		
43									End of test hole boring at 135 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
44											
45											

## WSDOT GEOTECH DIVISION

Operator: Brian Hiltz  
 Sounding: CPT-58  
 Elevation: 13.14

CPT Date/Time: 09-25-03 09:32  
 Location: 18+76.75 23.67 L  
 Job Number: XL-1640



## **APPENDIX D - LABORATORY TESTING**

## **SUMMARY OF LABORATORY TESTING**

Laboratory testing was performed on selected samples from the field exploration program. The samples are grouped into two categories, disturbed and undisturbed. Disturbed samples are those that were obtained during the Standard Penetration Test while undisturbed samples are those samples that were obtained using the WSDOT sampler.

All disturbed soil samples were visually examined and then grouped together based on particle size distribution, consistency, and color. Once groups of samples were established that had similar characteristics, a minimum of one sample per group was tested. The testing consisted of performing particle size analyses, determining the liquid limit if applicable, and determining the plastic limit and plasticity index if applicable. Specialized testing for the project included 2 unconsolidated undrained triaxial shear tests. The tests were done in accordance with AASHTO T-88, T-89, T-90, and T234 guide specifications respectively. After the testing was complete, the samples were classified using the Unified Soil Classification System (USCS).

Job No. XL-1640

Hole No. TH-3-03

**West Hoquiam Bridge - Maintenance-Turnout**

Date November 19, 2003

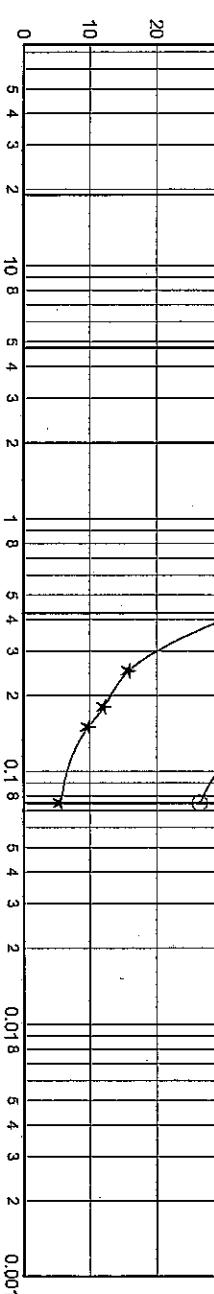
**Laboratory Summary**Washington State  
Department of Transportation**Project****Project**

Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MCC%	LL	PL	PI
● 9.5	2.90	D-2	ML	See Boring Log	SILT with SAND with organic	71			
☒ 38.5	11.73	D-11	SM	See Boring Log	SILTY SAND with organic	58			
▲ 43.5	13.26	D-12	SM	See Boring Log	SILTY SAND	56			
★ 48.5	14.78	D-13	SP-SM	See Boring Log	POORLY GRADED SAND with SILT and organic and wood chunks	39			
○ 53.5	16.31	D-14	SM	See Boring Log	SILTY SAND with organic and shells	45			

**GRADATION FRACTIONS**

US Sieve Opening In Inches | US Sieve Numbers

Hydrometer Analysis

**GRADATION VALUES****GRADATION VALUES**

	D60	D50	D30	D20	D10
●					
☒	0.100	0.08			
▲	0.106	0.09			
★	0.840	0.65	0.39	0.28	0.152
○	0.209	0.19	0.09		

Gravel

Sand

Coarse

Medium

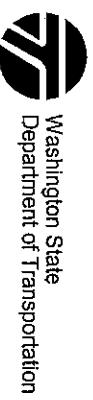
Fine

Silt and Clay

Job No. **XL-1640**  
Hole No. **TH-3-03**

Date **November 19, 2003**  
Sheet **2** of **3**

### Laboratory Summary



Washington State  
Department of Transportation

### Project **West Hoquiam Bridge - Maintenance-Turnout**

Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
● 58.5	17.83	D-15	SM	See Boring Log	SILTY SAND with organic	37			
☒ 63.5	19.35	D-16	ML	See Boring Log	SANDY SILT	58			
▲ 73.5	22.40	D-18	SM	See Boring Log	SILTY SAND	31			
★ 78.5	23.93	D-19	ML	See Boring Log	SANDY SILT with organic	56			
○ 83.5	25.45	D-20	SM	See Boring Log	SILTY SAND	42			

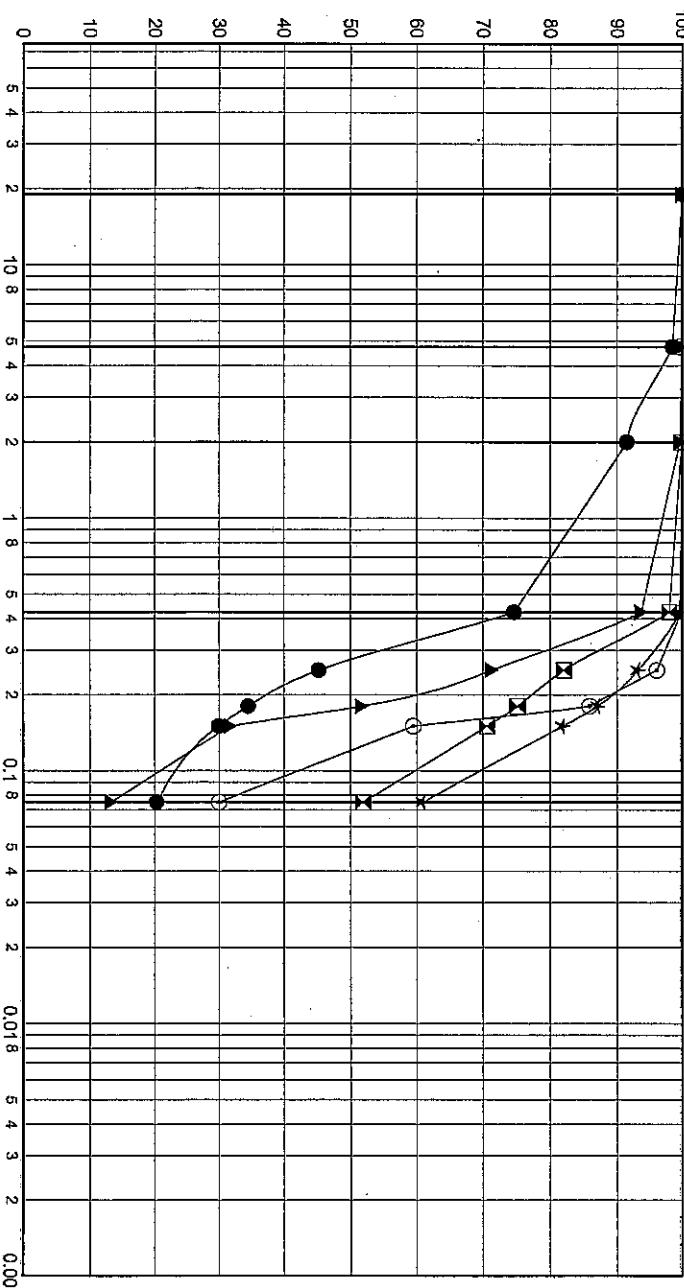
### GRADATION FRACTIONS

US Sieve Opening In Inches

US Sieve Numbers

Hydrometer Analysis

Percent Finer By Weight



### GRADATION VALUES

D60	D50	D30	D20	D10
● 0.326	0.27	0.15		
☒ 0.101				
▲ 0.207	0.18	0.14	0.10	
★				
○ 0.151	0.12	0.08		

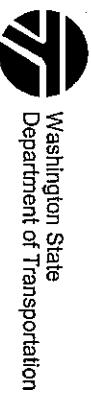
### Grain Size In Millimeter

	Gravel	Sand	Slit and Clay
	Coarse	Medium	Fine

Job No. XL-1640  
Hole No. TH-3-03

Date November 19, 2003  
Sheet 3 of 3

### Laboratory Summary



### Project West Hoquiam Bridge - Maintenance-Turnout

Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
● 88.5	26.97	D-21	SM	See Boring Log	SILTY SAND with organic	34			
☒ 93.5	28.50	D-22	SP-SM	See Boring Log	POORLY GRADED SAND with SILT	25			
▲ 104.7	31.91	U-24D	CH	See Boring Log	FAT CLAY with organic	63	346	42	304
* 133.5	40.69	D-27	GW	See Boring Log	WELL-GRADED GRAVEL with SAND	8			

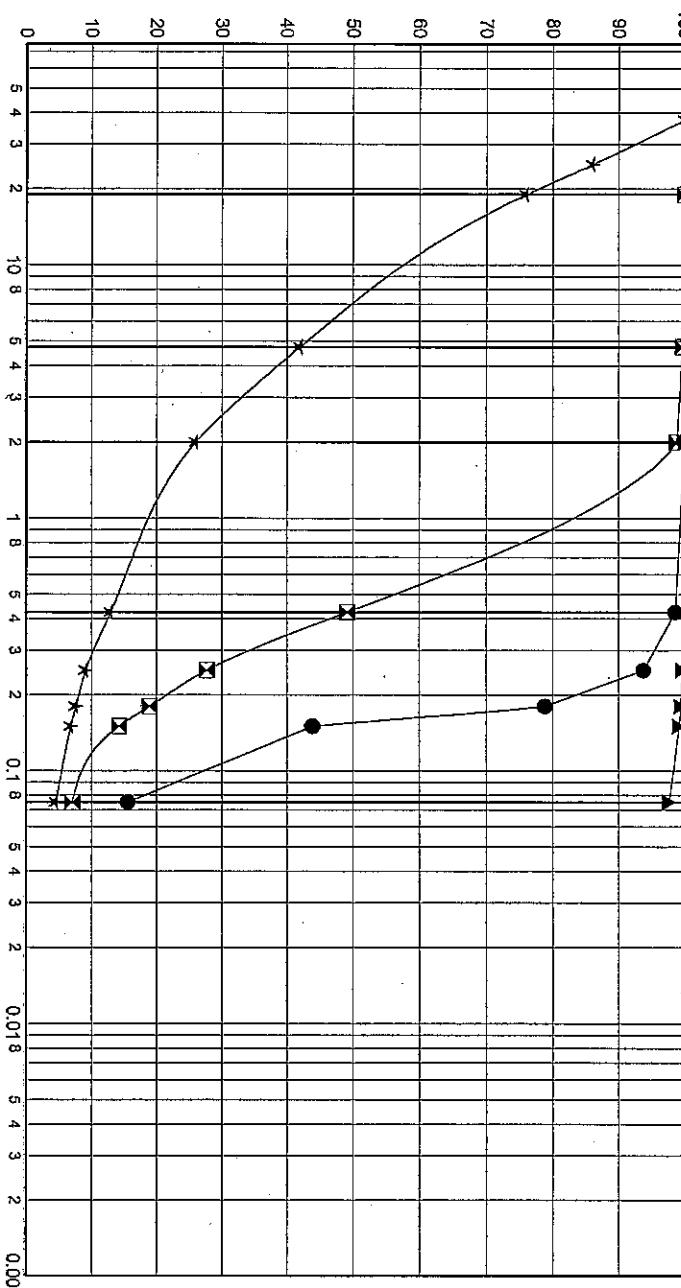
### GRADATION FRACTIONS

US Sieve Opening In Inches

US Sieve Numbers

Hydrometer Analysis

Percent Finer By Weight

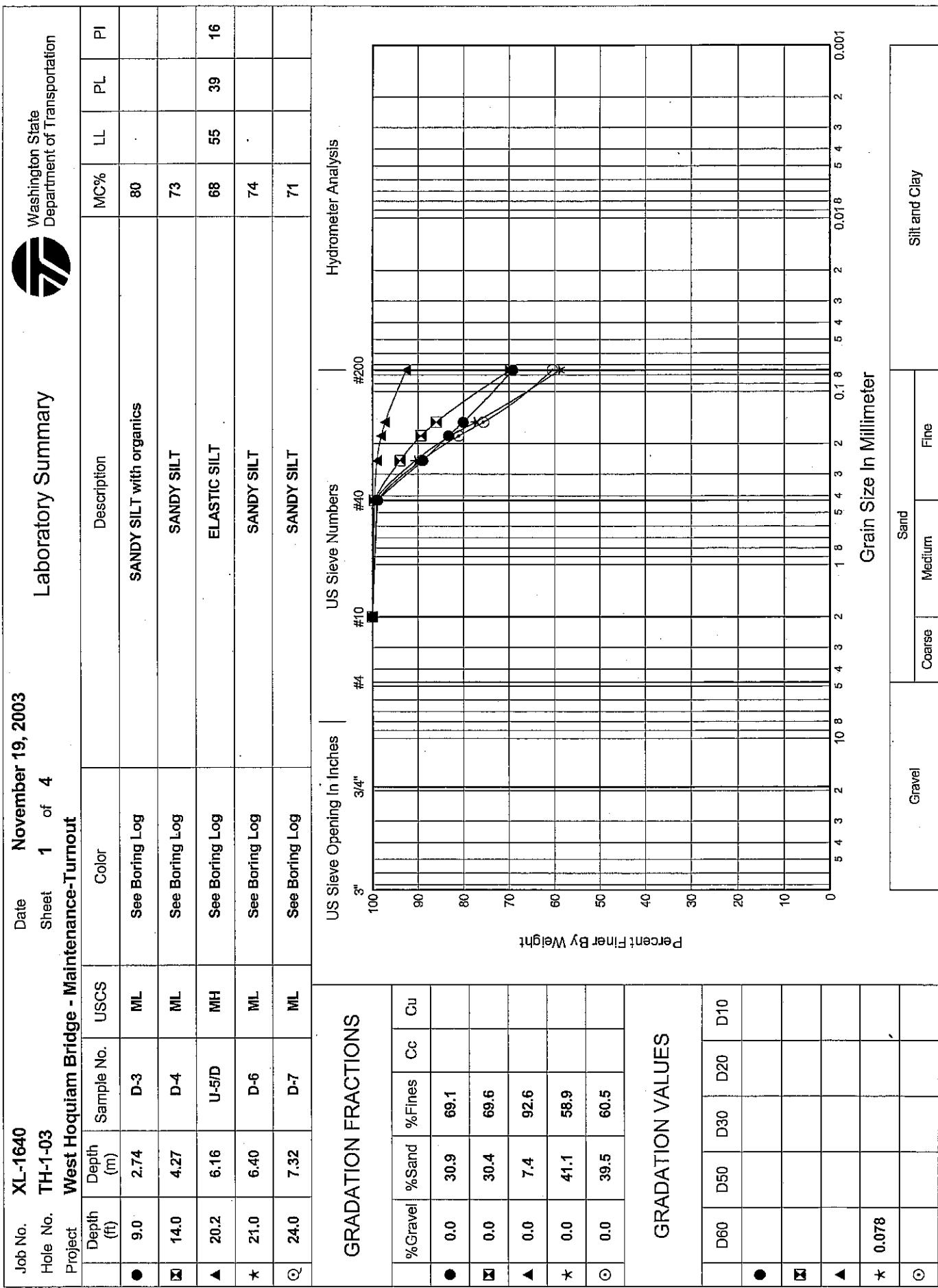


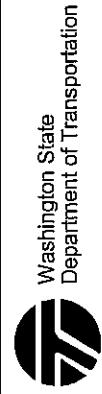
### GRADATION VALUES

D60 D50 D30 D20 D10

	D60	D50	D30	D20	D10
● 0.163	0.15	0.11	0.08		
☒ 0.596	0.44	0.26	0.19	0.100	
▲					
* 9.885	6.57	2.47	0.97	0.282	

	Gravel	Sand	Silt and Clay
	Coarse	Medium	Fine



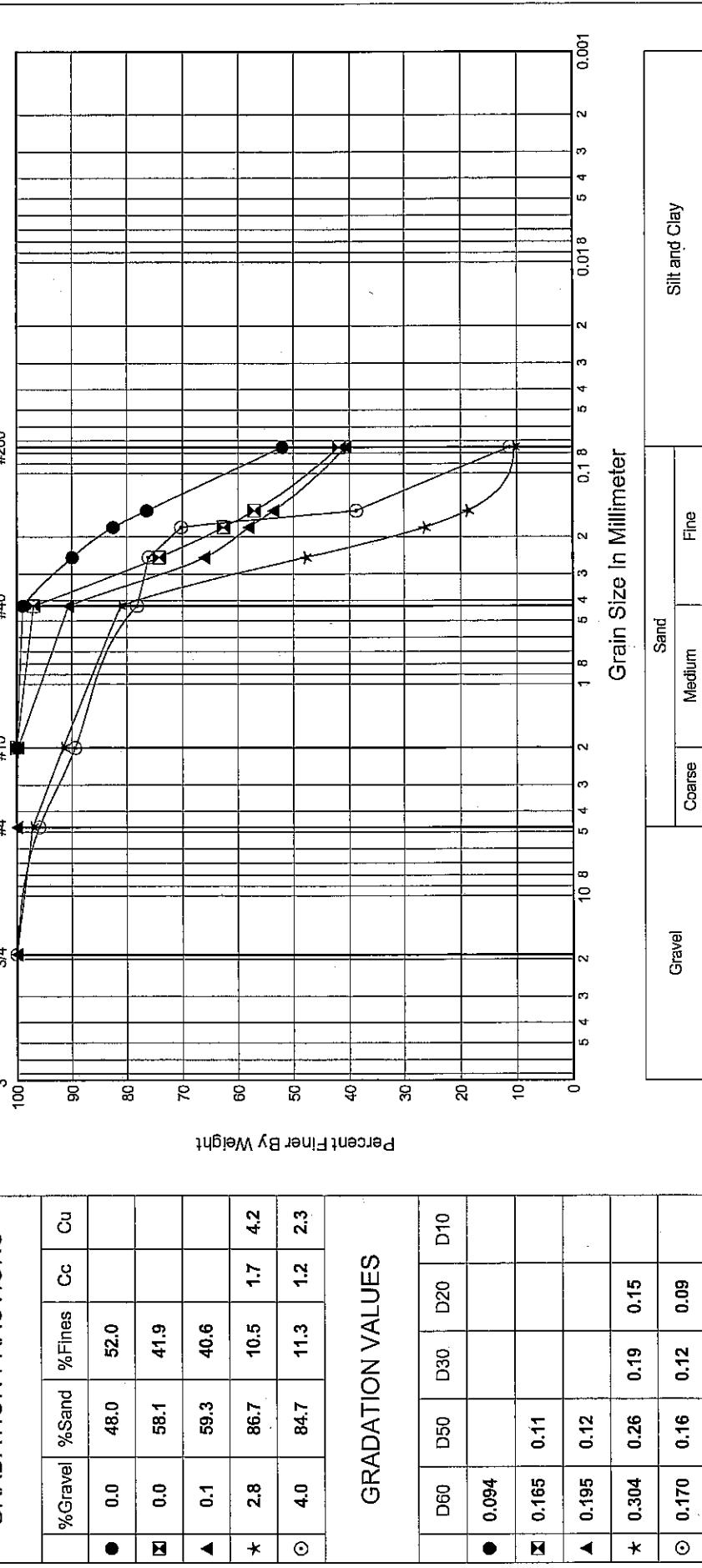


Job No. XL-1640 Date November 19, 2003  
 Hole No. TH-1-03 Sheet 2 of 4

**Project West Hoquiam Bridge - Maintenance-Turnout**

	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description	MC%	LL	PL	PI
●	34.0	10.36	D-9	ML	See Boring Log	SANDY SILT with organics	60			
▣	41.0	12.50	D-11	SM	See Boring Log	SILTY SAND	54			
▲	44.0	13.41	D-12	SM	See Boring Log	SILTY SAND with organics	53			
★	59.0	17.98	D-15	SP-SM	See Boring Log	POORLY GRADED SAND with SILT	29			
○	89.0	27.13	D-21	SP-SM	See Boring Log	POORLY GRADED SAND with SILT and wood chunks	43			

**GRADATION FRACTIONS**



Job No. XL-1640  
Hole No. TH-1-03  
Project West Hoquiam Bridge - Maintenance-Turnout

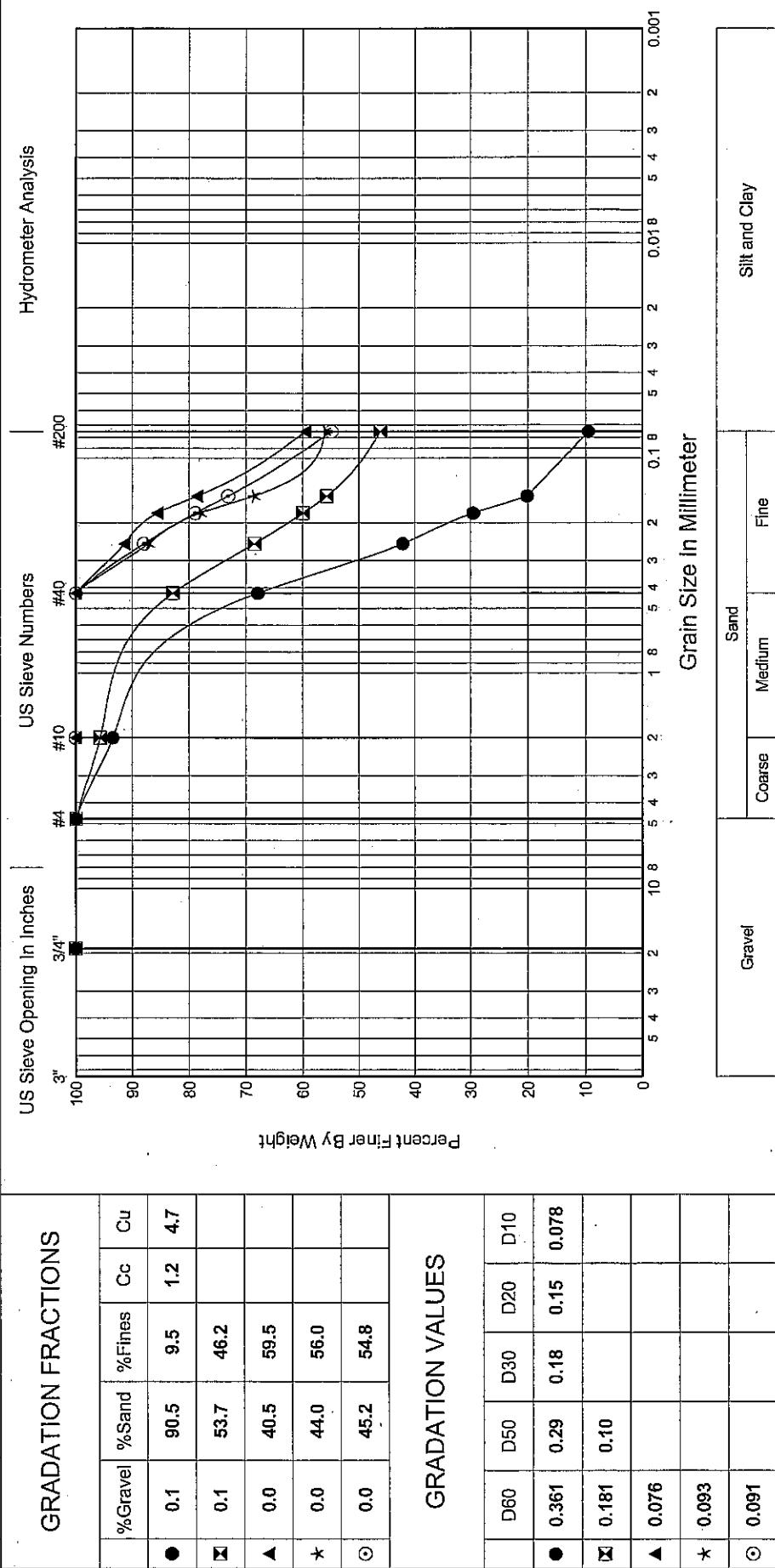
Date November 19, 2003

Sheet 3 of 4

## Laboratory Summary



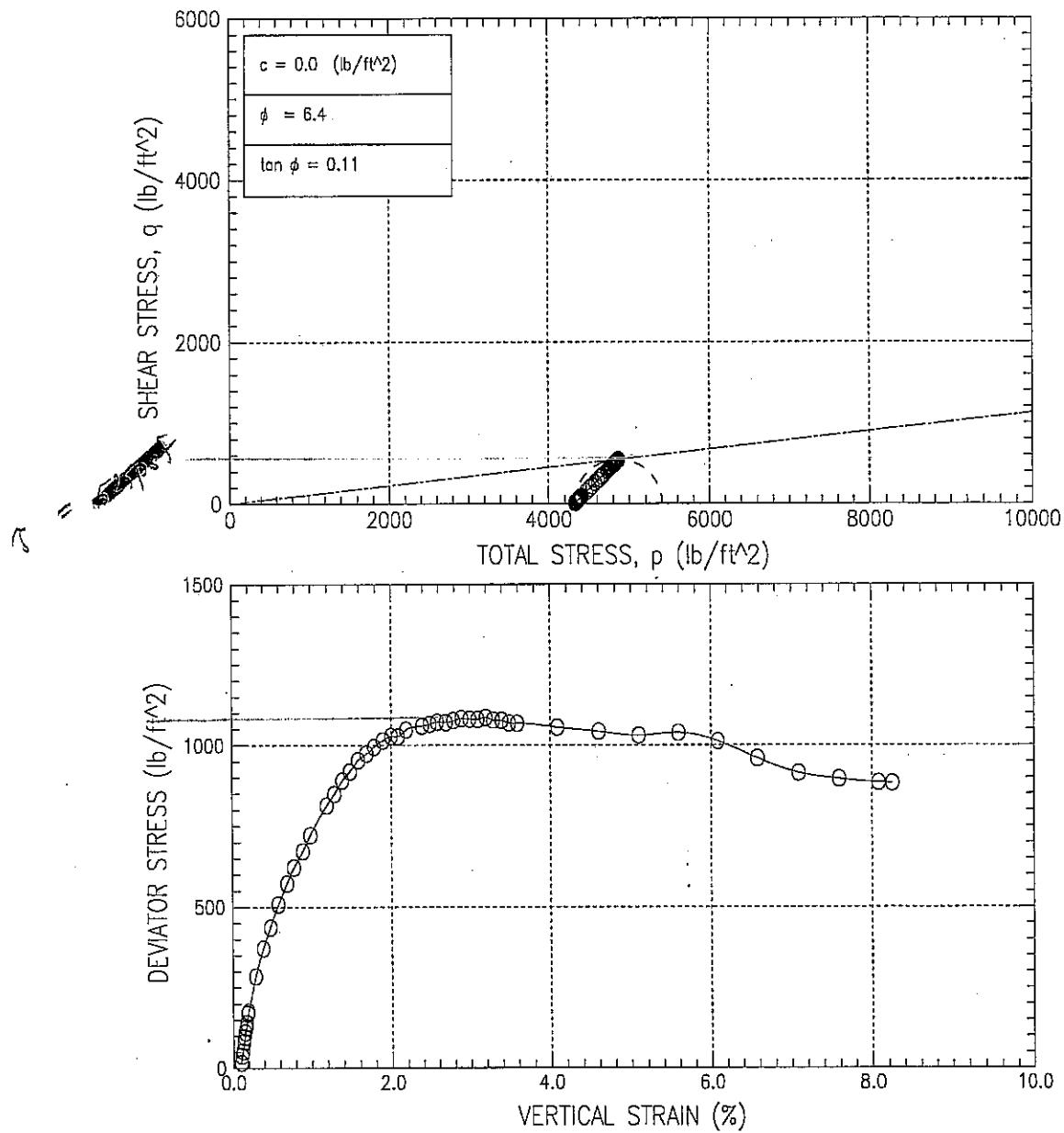
	Depth (ft)	Depth (m)	Sample No.	USCS	Color	Description
●	94.0	28.65	D-22	SP-SM	See Boring Log	POORLY GRADED SAND with SILT and organics and wood chunks
▣	99.0	30.18	D-23	SM	See Boring Log	SILTY SAND
▲	104.0	31.70	D-24	ML	See Boring Log	SANDY SILT
*	114.0	34.75	D-26	ML	See Boring Log	SANDY SILT
○	119.0	36.27	D-27	ML	See Boring Log	SANDY SILT





## **TRIAXIAL TEST RESULTS**

### UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



Washington State D.O.T.

Project Name : W. HOUQUAM BR. MAINT

Project No : XL-1640

Boring No : H-3-03

Sample No : U-24/D

Test Date : 11/5/03

Test No : 502824D

Depth : 104.7 ft

Description : MOIST DARK GRAY SILT

Remarks :

*Unit #3*

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT      Location : SR-101  
 Project No. : XL-1640      Test No. : 502824D  
 Boring No. : H-3-03      Test Date : 11/5/03      Tested by : LHB  
 Sample No. : U-24/D      Depth : 104.7 FT      Checked by : DVJ  
 Sample Type : WSDOT TUBE      Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

Height : 4.000 (in)      Piston Diameter : 0.625 (in)      Filter Correction : 0.00 (lb/ft<sup>2</sup>)  
 Diameter : 1.910 (in)      Piston Friction : 0.00 (lb)      Membrane Correction : 3.20 (lb/in)  
 Volume : 11.46 (in<sup>3</sup>)      Piston Weight : 916.60 (gm)      Area Correction : Uniform

TIME (min.)	STRAIN (%)	TOTAL	TOTAL	TOTAL		
		VERTICAL	VERTICAL	HORIZONTAL	P (lb/ft <sup>2</sup> )	q (lb/ft <sup>2</sup> )
1)	0	0.10	4336.77	4320.01	4328.39	8.38
2)	0.016666	0.11	4358.55	4320.01	4339.28	19.27
3)	0.033333	0.12	4373.07	4320.01	4346.54	26.53
4)	0.066666	0.14	4402.11	4320.01	4361.06	41.05
5)	0.083333	0.15	4416.62	4320.01	4368.32	48.31
6)	0.116667	0.15	4431.14	4320.01	4375.57	55.56
7)	0.133333	0.17	4445.64	4320.01	4382.82	62.81
8)	0.15	0.18	4460.15	4320.01	4390.08	70.07
9)	0.183333	0.19	4489.15	4320.01	4404.58	84.57
10)	0.2	0.20	4496.40	4320.01	4408.20	88.19
11)	0.4	0.29	4604.98	4320.01	4462.49	142.48
12)	0.616667	0.39	4691.59	4320.01	4505.80	185.79
13)	0.8	0.49	4756.36	4320.01	4538.19	218.17
14)	1.01667	0.59	4828.19	4320.01	4574.10	254.09
15)	1.23333	0.70	4892.61	4320.01	4606.31	286.30
16)	1.43333	0.79	4942.60	4320.01	4631.31	311.30
17)	1.65	0.90	4992.36	4320.01	4656.18	336.17
18)	1.85	0.99	5042.10	4320.01	4681.06	361.05
19)	2.28333	1.20	5134.01	4320.01	4727.01	407.00
20)	2.48333	1.29	5169.11	4320.01	4744.56	424.55
21)	2.7	1.39	5211.25	4320.01	4765.63	445.62
22)	2.88333	1.49	5239.04	4320.01	4779.53	459.52
23)	3.1	1.59	5273.86	4320.01	4796.94	476.93
24)	3.3	1.69	5294.32	4320.01	4807.16	487.15
25)	3.48333	1.79	5314.80	4320.01	4817.41	497.39
26)	3.7	1.90	5335.09	4320.01	4827.55	507.54
27)	3.9	2.00	5348.28	4320.01	4834.15	514.14
28)	4.08333	2.09	5347.36	4320.01	4833.69	513.68
29)	4.3	2.19	5367.63	4320.01	4843.82	523.81
30)	4.7	2.39	5379.63	4320.01	4849.82	529.81
31)	4.9	2.49	5385.69	4320.01	4852.85	532.84
32)	5.08333	2.58	5391.74	4320.01	4855.87	535.86
33)	5.3	2.69	5390.53	4320.01	4855.27	535.26
34)	5.48333	2.79	5396.56	4320.01	4858.28	538.27

35)	5.68333	2.89	5402.49	4320.01	4861.25	541.24
36)	5.88333	2.99	5401.35	4320.01	4860.68	540.67

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT      Location : SR-101  
 Project No. : XL-1640      Test No. : 502824D  
 Boring No. : H-3-03      Test Date : 11/5/03      Tested by : LHB  
 Sample No. : U-24/D      Depth : 104.7 FT      Checked by : DVJ  
 Sample Type : WSDOT TUBE      Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

TIME (min.)	STRAIN (%)	TOTAL VERTICAL	TOTAL VERTICAL	HORIZONTAL	TOTAL	q (lb/ft^2)
		STRESS (lb/ft^2)	STRESS (lb/ft^2)	P (lb/ft^2)	q (lb/ft^2)	
37)	6.08333	3.10	5400.21	4320.01	4860.11	540.10
38)	6.28333	3.20	5406.11	4320.01	4863.06	543.05
39)	6.5	3.29	5398.01	4320.01	4859.01	539.00
40)	6.7	3.39	5396.87	4320.01	4858.44	538.43
41)	6.9	3.49	5388.80	4320.01	4854.40	534.39
42)	7.1	3.59	5387.66	4320.01	4853.84	533.83
43)	8.08333	4.09	5375.19	4320.01	4847.60	527.59
44)	9.1	4.60	5362.64	4320.01	4841.32	521.31
45)	10.1167	5.10	5350.31	4320.01	4835.16	515.15
46)	11.1167	5.59	5358.65	4320.01	4839.33	519.32
47)	12.1167	6.09	5332.70	4320.01	4826.36	506.35
48)	13.1333	6.58	5279.80	4320.01	4799.91	479.90
49)	14.1	7.09	5234.10	4320.01	4777.05	457.04
50)	15.0833	7.59	5215.70	4320.01	4767.86	447.85
51)	16.05	8.08	5204.28	4320.01	4762.14	442.13
52)	16.3833	8.25	5202.66	4320.01	4761.34	441.33

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT

Location : SR-101

Project No. : XL-1640

Test No. : 502824D

Boring No. : H-3-03

Test Date : 11/5/03

Tested by : LHB

Sample No. : U-24/D

Depth : 104.7 FT

Checked by : DVJ

Sample Type : WSDOT TUBE

Elevation :

Soil Description : MOIST DARK GRAY SILT

Remarks :

Height : 4.000 (in)

Piston Diameter : 0.625 (in)

Filter Correction : 0.00 (lb/ft^2)

Diameter : 1.910 (in)

Piston Friction : 0.00 (lb)

Membrane Correction : 3.20 (lb/in)

Volume : 11.46 (in^3)

Piston Weight : 916.60 (gm)

Area Correction : Uniform

TIME (min.)	CHANGE IN LENGTH (in)	VERTICAL STRAIN (%)	CORR. AREA (in^2)	DEV. LOAD (lb)	CORR. DEV. LOAD (lb)	DEV. STRESS (lb/ft^2)	VERTICAL STRESS (lb/ft^2)
1)	0 0.004	0.10	2.87	7.52	0.33	16.76	4336.77
2)	0.016666 0.004	0.11	2.87	7.96	0.77	38.54	4358.55
3)	0.033333 0.005	0.12	2.87	8.25	1.06	53.06	4373.07
4)	0.066666 0.006	0.14	2.87	8.83	1.64	82.10	4402.11
5)	0.083333 0.006	0.15	2.87	9.11	1.93	96.61	4416.62
6)	0.116667 0.006	0.15	2.87	9.40	2.21	111.13	4431.14
7)	0.133333 0.007	0.17	2.87	9.69	2.50	125.63	4445.64
8)	0.15 0.007	0.18	2.87	9.98	2.79	140.13	4460.15
9)	0.183333 0.008	0.19	2.87	10.56	3.37	169.14	4489.15
10)	0.2 0.008	0.20	2.87	10.71	3.52	176.39	4496.40
11)	0.4 0.012	0.29	2.87	12.88	5.69	284.97	4504.98
12)	0.616667 0.016	0.39	2.88	14.61	7.42	371.58	4691.59
13)	0.8 0.020	0.49	2.88	15.91	8.72	436.35	4756.36
14)	1.01667 0.024	0.59	2.88	17.36	10.17	508.18	4828.19
15)	1.23333 0.028	0.70	2.89	18.66	11.47	572.60	4892.61
16)	1.43333 0.032	0.79	2.89	19.68	12.49	622.59	4942.60
17)	1.65 0.036	0.90	2.89	20.69	13.50	672.35	4992.36
18)	1.85 0.040	0.99	2.89	21.70	14.51	722.09	5042.10
19)	2.28333 0.048	1.20	2.90	23.58	16.39	813.99	5134.01
20)	2.48333 0.052	1.29	2.90	24.31	17.12	849.10	5169.11
21)	2.7 0.056	1.39	2.91	25.17	17.98	891.24	5211.25
22)	2.88333 0.060	1.49	2.91	25.75	18.56	919.03	5239.04
23)	3.1 0.064	1.59	2.91	26.48	19.29	953.85	5273.86
24)	3.3 0.068	1.69	2.91	26.91	19.72	974.31	5294.32
25)	3.48333 0.072	1.79	2.92	27.34	20.15	994.79	5314.80
26)	3.7 0.076	1.90	2.92	27.78	20.59	1015.08	5335.09
27)	3.9 0.080	2.00	2.92	28.07	20.88	1028.27	5348.28
28)	4.08333 0.084	2.09	2.93	28.07	20.88	1027.35	5347.36
29)	4.3 0.088	2.19	2.93	28.50	21.31	1047.62	5367.63
30)	4.7 0.096	2.39	2.94	28.79	21.60	1059.62	5379.63
31)	4.9 0.100	2.49	2.94	28.93	21.75	1065.68	5385.69
32)	5.08333 0.103	2.58	2.94	29.08	21.89	1071.73	5391.74
33)	5.3 0.108	2.69	2.94	29.08	21.89	1070.52	5390.53
34)	5.48333 0.112	2.79	2.95	29.22	22.03	1076.55	5396.56

35)	5.68333	0.116	2.89	2.95	29.37	22.18	1082.48	5402.49
36)	5.88333	0.120	2.99	2.95	29.37	22.18	1081.34	5401.35

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT Location : SR-101  
 Project No. : XL-1640 Test No. : 502824D  
 Boring No. : H-3-03 Test Date : 11/5/03 Tested by : LHB  
 Sample No. : U-24/D Depth : 104.7 FT Checked by : DVJ  
 Sample Type : WSDOT TUBE Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

	CHANGE		VERTICAL	CORR.	DEV.	CORR. DEV.	DEV.	VERTICAL
	TIME	IN LENGTH	STRAIN	AREA	LOAD	LOAD	STRESS	STRESS
	(min.)	(in)	(%)	(in <sup>2</sup> )	(lb)	(lb)	(lb/ft <sup>2</sup> )	(lb/ft <sup>2</sup> )
37)	6.08333	0.124	3.10	2.96	29.37	22.18	1080.20	5400.21
38)	6.28333	0.128	3.20	2.96	29.51	22.32	1086.10	5406.11
39)	6.5	0.132	3.29	2.96	29.37	22.18	1078.00	5398.01
40)	6.7	0.136	3.39	2.97	29.37	22.18	1076.86	5396.87
41)	6.9	0.140	3.49	2.97	29.22	22.03	1068.79	5388.80
42)	7.1	0.144	3.59	2.97	29.22	22.03	1067.65	5387.66
43)	8.08333	0.164	4.09	2.99	29.08	21.89	1055.18	5375.19
44)	9.1	0.184	4.60	3.00	28.93	21.75	1042.62	5362.64
45)	10.1167	0.204	5.10	3.02	28.79	21.60	1030.30	5350.31
46)	11.1167	0.224	5.59	3.03	29.08	21.89	1038.64	5358.65
47)	12.1167	0.244	6.09	3.05	28.65	21.46	1012.69	5332.70
48)	13.1333	0.263	6.58	3.07	27.63	20.44	959.79	5279.80
49)	14.1	0.284	7.09	3.08	26.76	19.58	914.08	5234.10
50)	15.0833	0.304	7.59	3.10	26.48	19.29	895.69	5215.70
51)	16.05	0.323	8.08	3.12	26.33	19.14	884.27	5204.28
52)	16.3833	0.330	8.25	3.12	26.33	19.14	882.65	5202.66

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UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT      Location : SR-101  
Project No. : XL-1640      Test No. : 502824D  
Boring No. : H-3-03      Test Date : 11/5/03      Tested by : LHB  
Sample No. : U-24/D      Depth : 104.7 FT      Checked by : DVJ  
Sample Type : WSDOT TUBE      Elevation :  
Soil Description : MOIST DARK GRAY SILT  
Remarks :

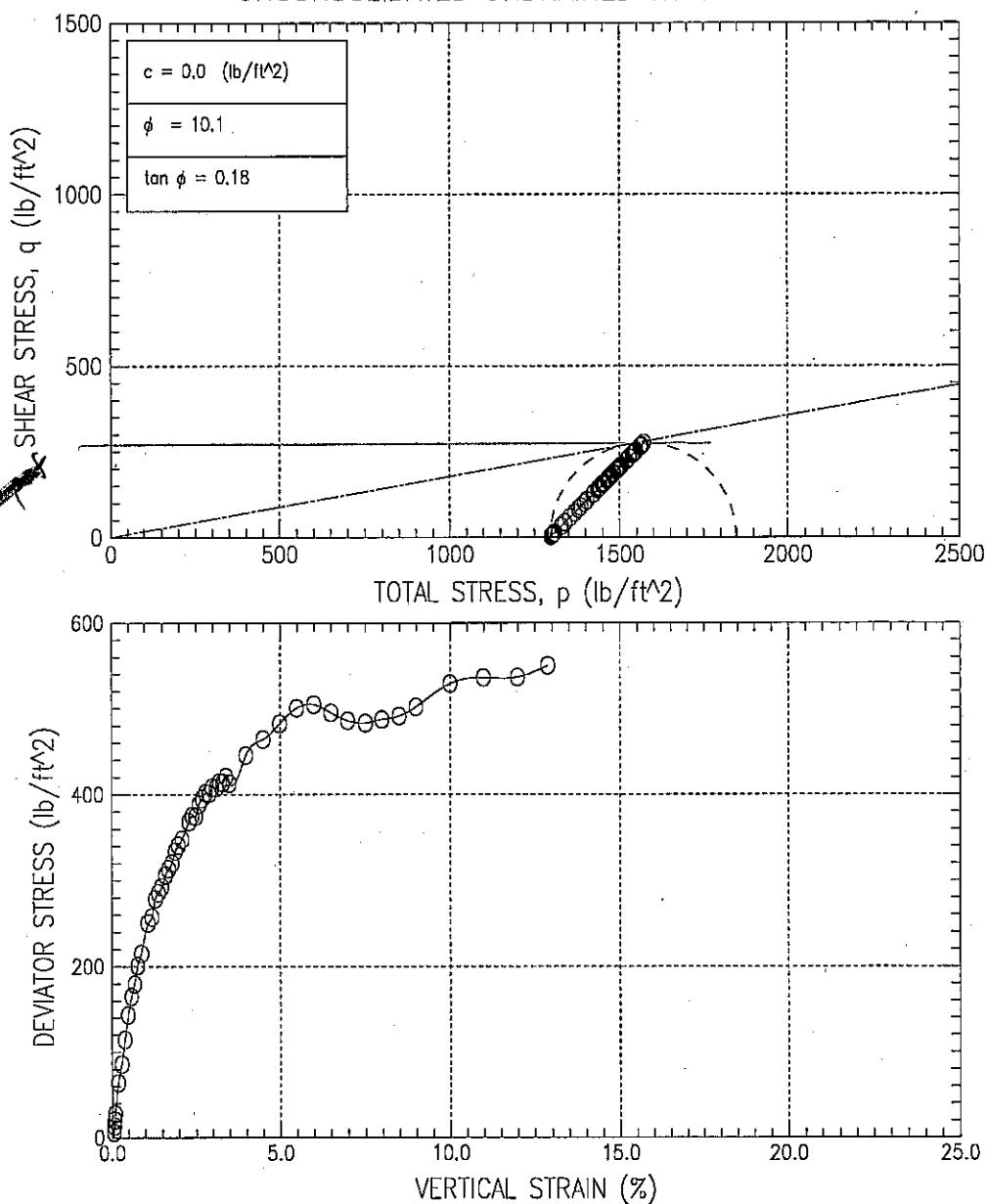
Liquid Limit : 0      Plastic Limit : 0      Specific Gravity : 2.65

WATER CONTENT  
BEFORE TEST      AFTER TEST

CONTAINER NO.	BEFORE TEST	AFTER TEST
WT CONTAINER + WET SOIL (gm)	0.00	0.00
WT CONTAINER + DRY SOIL (gm)	0.00	0.00
WT WATER (gm)	0.00	0.00
WT CONTAINER (gm)	0.00	0.00
WT DRY SOIL (gm)	0.00	0.00
WATER CONTENT (%)	0.00	0.00

Maximum Shear Stress = 543.05 (lb/ft<sup>2</sup>) at a Vertical Strain of 3.20 %

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



Washington State D.O.T.

Project Name : W. HOUQUIAM BR. MAINT

Project No : XL-1640

Boring No : H-1-03

Sample No : U-5/D

Test Date : 11/5/03

Test No : 50295D

Depth : 20.2 FT

Description : MOIST DARK GRAY SILT

Remarks :

Unit # 1

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT      Location : SR-101  
 Project No. : XL-1640      Test No. : 50295D  
 Boring No. : H-1-03      Test Date : 11/5/03      Tested by : LHB  
 Sample No. : U-5/D      Depth : 20.2 FT      Checked by : DVJ  
 Sample Type : WSDOT TUBE      Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

Height : 4.000 (in)      Piston Diameter : 0.625 (in)      Filter Correction : 0.00 (lb/ft^2)  
 Diameter : 1.910 (in)      Piston Friction : 0.00 (lb)      Membrane Correction : 3.20 (lb/in)  
 Volume : 11.46 (in^3)      Piston Weight : 914.90 (gm)      Area Correction : Uniform

TIME (min.)	STRAIN (%)	TOTAL	TOTAL	TOTAL		
		VERTICAL	VERTICAL	HORIZONTAL	P (lb/ft^2)	q (lb/ft^2)
1)	0	0.00	1258.51	1296.00	1277.25	-18.75
2)	0.016666	0.01	1258.51	1296.00	1277.25	-18.75
3)	0.05	0.02	1280.32	1296.00	1288.16	-7.84
4)	0.066666	0.03	1287.59	1296.00	1291.80	-4.20
5)	0.083333	0.04	1287.59	1296.00	1291.80	-4.20
6)	0.116667	0.05	1302.13	1296.00	1299.06	3.06
7)	0.133333	0.07	1302.13	1296.00	1299.06	3.06
8)	0.15	0.07	1309.39	1296.00	1302.70	6.70
9)	0.166667	0.08	1316.66	1296.00	1306.33	10.33
10)	0.2	0.09	1323.92	1296.00	1309.96	13.96
11)	0.416667	0.19	1360.18	1296.00	1328.09	32.09
12)	0.616667	0.29	1381.86	1296.00	1338.93	42.93
13)	0.816667	0.39	1410.75	1296.00	1353.38	57.38
14)	1.03333	0.49	1439.58	1296.00	1367.79	71.79
15)	1.23333	0.59	1461.11	1296.00	1378.56	82.56
16)	1.43333	0.69	1475.40	1296.00	1385.70	89.70
17)	1.63333	0.78	1496.87	1296.00	1396.43	100.43
18)	1.85	0.89	1511.06	1296.00	1403.53	107.53
19)	2.25	1.09	1546.59	1296.00	1421.30	125.30
20)	2.48333	1.20	1553.50	1296.00	1424.75	128.75
21)	2.68333	1.30	1574.76	1296.00	1435.38	139.38
22)	2.86667	1.39	1581.68	1296.00	1438.84	142.84
23)	3.06667	1.49	1588.55	1296.00	1442.28	146.28
24)	3.28333	1.60	1602.54	1296.00	1449.27	153.27
25)	3.46667	1.69	1609.41	1296.00	1452.71	156.71
26)	3.66667	1.79	1616.23	1296.00	1456.11	160.11
27)	3.86667	1.89	1630.16	1296.00	1463.08	167.08
28)	4.05	1.98	1636.99	1296.00	1466.49	170.49
29)	4.26667	2.09	1643.73	1296.00	1469.86	173.86
30)	4.68333	2.29	1664.31	1296.00	1480.16	184.16
31)	4.88333	2.39	1671.05	1296.00	1483.53	187.53
32)	5.1	2.50	1670.63	1296.00	1483.32	187.32
33)	5.28333	2.59	1684.43	1296.00	1490.22	194.22
34)	5.48333	2.69	1691.13	1296.00	1493.57	197.57

35)	5.68333	2.79	1697.79	1296.00	1496.89	200.89
36)	5.88333	2.89	1697.36	1296.00	1496.68	200.68

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT Location : SR-101  
 Project No. : XL-1640 Test No. : 50295D  
 Boring No. : H-1-03 Test Date : 11/5/03 Tested by : LHB  
 Sample No. : U-5/D Depth : 20.2 FT Checked by : DVJ  
 Sample Type : WSDOT TUBE Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

TIME (min.)	STRAIN (%)	TOTAL VERTICAL	TOTAL VERTICAL	TOTAL HORIZONTAL	TOTAL P (lb/ft^2)	q (lb/ft^2)
		STRESS (lb/ft^2)	STRESS (lb/ft^2)	P (lb/ft^2)		
37)	6.06667	2.98	1704.06	1296.00	1500.03	204.03
38)	6.28333	3.10	1703.56	1296.00	1499.78	203.78
39)	6.48333	3.20	1710.17	1296.00	1503.09	207.09
40)	6.68333	3.29	1709.77	1296.00	1502.88	206.88
41)	6.88333	3.39	1716.39	1296.00	1506.19	210.19
42)	7.08333	3.49	1708.92	1296.00	1502.46	206.46
43)	8.06667	3.98	1741.74	1296.00	1518.87	222.87
44)	9.08333	4.49	1760.20	1296.00	1528.10	232.10
45)	10.1	4.99	1778.51	1296.00	1537.26	241.26
46)	11.1167	5.49	1796.57	1296.00	1546.29	250.29
47)	12.1333	5.99	1800.74	1296.00	1548.37	252.37
48)	13.15	6.50	1791.24	1296.00	1543.62	247.62
49)	14.1667	6.99	1781.88	1296.00	1538.94	242.94
50)	15.15	7.50	1779.21	1296.00	1537.61	241.61
51)	16.1333	7.98	1783.39	1296.00	1539.69	243.69
52)	17.15	8.49	1787.33	1296.00	1541.67	245.67
53)	18.15	8.99	1797.90	1296.00	1546.95	250.95
54)	20.15	10.00	1825.03	1296.00	1560.51	264.51
55)	22.1333	10.99	1832.18	1296.00	1564.09	268.09
56)	24.1333	11.99	1832.55	1296.00	1564.28	268.28
57)	25.85	12.88	1846.13	1296.00	1571.06	275.06

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT Location : SR-101  
 Project No. : XL-1640 Test No. : 50295D  
 Boring No. : H-1-03 Test Date : 11/5/03 Tested by : LHB  
 Sample No. : U-5/D Depth : 20.2 FT Checked by : DVJ  
 Sample Type : WSDOT TUBE Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

Height : 4.000 (in) Piston Diameter : 0.625 (in) Filter Correction : 0.00 (lb/ft^2)  
 Diameter : 1.910 (in) Piston Friction : 0.00 (lb) Membrane Correction : 3.20 (lb/in)  
 Volume : 11.46 (in^3) Piston Weight : 914.90 (gm) Area Correction : Uniform

	CHANGE	VERTICAL	CORR.	DEV.	CORR. DEV.	DEV.	VERTICAL
	TIME IN LENGTH	STRAIN	AREA	LOAD	LOAD	STRESS	STRESS
	(min.)	(in)	(%)	(in^2)	(lb)	(lb)	(lb/ft^2)
1)		0 0.000	0.00	2.87	0.00	-0.75	-37.49
2)	0.016666	0.000	0.01	2.87	0.00	-0.75	-37.49
3)	0.05	0.001	0.02	2.87	0.43	-0.31	-15.68
4)	0.066666	0.001	0.03	2.87	0.58	-0.17	-8.41
5)	0.083333	0.001	0.04	2.87	0.58	-0.17	-8.41
6)	0.116667	0.002	0.05	2.87	0.87	0.12	6.13
7)	0.133333	0.003	0.07	2.87	0.87	0.12	6.13
8)	0.15	0.003	0.07	2.87	1.01	0.27	13.39
9)	0.166667	0.003	0.08	2.87	1.16	0.41	20.66
10)	0.2	0.004	0.09	2.87	1.30	0.56	27.92
11)	0.416667	0.008	0.19	2.87	2.03	1.28	64.18
12)	0.616667	0.012	0.29	2.87	2.46	1.71	85.86
13)	0.816667	0.015	0.39	2.88	3.04	2.29	114.75
14)	1.03333	0.020	0.49	2.88	3.62	2.87	143.58
15)	1.23333	0.024	0.59	2.88	4.05	3.30	165.11
16)	1.43333	0.027	0.69	2.89	4.34	3.59	179.40
17)	1.63333	0.031	0.78	2.89	4.77	4.03	200.87
18)	1.85	0.036	0.89	2.89	5.06	4.32	215.06
19)	2.25	0.044	1.09	2.90	5.79	5.04	250.59
20)	2.48333	0.048	1.20	2.90	5.93	5.19	257.50
21)	2.68333	0.052	1.30	2.90	6.37	5.62	278.76
22)	2.86667	0.055	1.39	2.91	6.51	5.76	285.68
23)	3.06667	0.060	1.49	2.91	6.66	5.91	292.55
24)	3.28333	0.064	1.60	2.91	6.94	6.20	306.54
25)	3.46667	0.067	1.69	2.91	7.09	6.34	313.41
26)	3.66667	0.072	1.79	2.92	7.23	6.49	320.23
27)	3.86667	0.076	1.89	2.92	7.52	6.78	334.16
28)	4.05	0.079	1.98	2.92	7.67	6.92	340.99
29)	4.26667	0.084	2.09	2.93	7.81	7.07	347.73
30)	4.68333	0.092	2.29	2.93	8.25	7.50	368.31
31)	4.88333	0.095	2.39	2.94	8.39	7.65	375.05
32)	5.1	0.100	2.50	2.94	8.39	7.65	374.63
33)	5.28333	0.104	2.59	2.94	8.68	7.93	388.43
34)	5.48333	0.107	2.69	2.94	8.83	8.08	395.13

35)	5.68333	0.112	2.79	2.95	8.97	8.22	401.79	1697.79
36)	5.88333	0.116	2.89	2.95	8.97	8.22	401.36	1697.36

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT

Location : SR-101

Project No. : XL-1640

Test No. : 50295D

Boring No. : H-1-03

Test Date : 11/5/03

Tested by : LHB

Sample No. : U-5/D

Depth : 20.2 FT

Checked by : DVJ

Sample Type : WSDOT TUBE

Elevation :

Soil Description : MOIST DARK GRAY SILT

Remarks :

		CHANGE	VERTICAL	CORR.	DEV.	CORR. DEV.	DEV.	VERTICAL
	TIME	IN LENGTH	STRAIN	AREA	LOAD	LOAD	STRESS	STRESS
	(min.)	(in)	(%)	(in^2)	(lb)	(lb)	(lb/ft^2)	(lb/ft^2)
37)	6.06667	0.119	2.98	2.95	9.11	8.37	408.06	1704.06
38)	6.28333	0.124	3.10	2.96	9.11	8.37	407.56	1703.56
39)	6.48333	0.128	3.20	2.96	9.26	8.51	414.17	1710.17
40)	6.68333	0.132	3.29	2.96	9.26	8.51	413.77	1709.77
41)	6.88333	0.135	3.39	2.97	9.40	8.66	420.39	1716.39
42)	7.08333	0.140	3.49	2.97	9.26	8.51	412.92	1708.92
43)	8.06667	0.159	3.98	2.98	9.98	9.24	445.74	1741.74
44)	9.08333	0.180	4.49	3.00	10.42	9.67	464.20	1760.20
45)	10.1	0.199	4.99	3.02	10.85	10.10	482.51	1778.51
46)	11.1167	0.220	5.49	3.03	11.28	10.54	500.57	1796.57
47)	12.1333	0.240	5.99	3.05	11.43	10.68	504.74	1800.74
48)	13.15	0.260	6.50	3.06	11.28	10.54	495.24	1791.24
49)	14.1667	0.279	6.99	3.08	11.14	10.39	485.88	1781.88
50)	15.15	0.300	7.50	3.10	11.14	10.39	483.21	1779.21
51)	16.1333	0.319	7.98	3.11	11.28	10.54	487.39	1783.39
52)	17.15	0.340	8.49	3.13	11.43	10.68	491.33	1787.33
53)	18.15	0.359	8.99	3.15	11.72	10.97	501.90	1797.90
54)	20.15	0.400	10.00	3.18	12.44	11.70	529.03	1825.03
55)	22.1333	0.439	10.99	3.22	12.73	11.99	536.18	1832.18
56)	24.1333	0.479	11.99	3.26	12.88	12.13	536.55	1832.55
57)	25.85	0.515	12.88	3.29	13.31	12.56	550.13	1846.13

## UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

Project : W. HOUQUIAM BR. MAINT      Location : SR-101  
 Project No. : XL-1640      Test No. : 50295D  
 Boring No. : H-1-03      Test Date : 11/5/03      Tested by : LHB  
 Sample No. : U-5/D      Depth : 20.2 FT      Checked by : DVJ  
 Sample Type : WSDOT TUBE      Elevation :  
 Soil Description : MOIST DARK GRAY SILT  
 Remarks :

Liquid Limit : 0      Plastic Limit : 0      Specific Gravity : 2.65

WATER CONTENT	
BEFORE TEST	AFTER TEST

CONTAINER NO.		
WT CONTAINER + WET SOIL (gm)	0.00	0.00
WT CONTAINER + DRY SOIL (gm)	0.00	0.00
WT WATER (gm)	0.00	0.00
WT CONTAINER (gm)	0.00	0.00
WT DRY SOIL (gm)	0.00	0.00
WATER CONTENT (%)	0.00	0.00

Maximum Shear Stress = 275.06 (lb/ft^2) at a Vertical Strain of 12.88 %



**Washington State  
Department of Transportation**

**FILE**

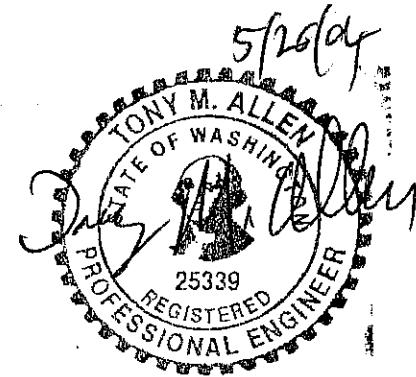
**Memorandum**

May 12, 2004

TO: J. Kapur/M. Anderson  
Bridge and Structures, MS47340  
*D.J.*

FROM: T.M. Allen/D.V. Jenkins  
EEP Geotechnical Branch, 47365

SUBJECT: US101, XL1640A  
Hoquiam River/Simpson Ave. Maintenance Turnout  
Addendum #1 – Geotechnical Report



**EXPIRES 07-01-05**

Recent technical discussion between staff of the Bridge and Structures office and the Geotechnical Division has resulted in us re-evaluating our foundation recommendations for support of the proposed maintenance turnout structure on the Hoquiam River and Simpson Avenue Bridge on US101. Our geotechnical report dated March 1, 2004 provided foundation recommendations using drilled shafts. Drilled shaft foundations were recommended based on our assumption the foundation design must account for high seismically induced downdrag loads in the extreme case. As a result of these recommendations, the total cost of the structure widening for the maintenance turnout was very high using a drilled shaft foundation. Medium to high capacity piles were not recommended because of our concerns regarding the potential for pile driving induced liquefaction that could result in settlement of the existing pile supported bridge.

We have re-evaluated our recommendations and are now providing a timber pile supported foundation option that we feel would meet design AASHTO requirements and would not cause pile driving induced liquefaction during construction. We must note that downdrag loads under a seismic condition have not been factored into this pile design. We feel the structure would not fail under a design seismic earthquake event but total settlement of the structure could be significant.

The existing structure is supported on low capacity timber piling. The approaches to the main span were widened in 1977 under contract 0498. This contract provide widening of each timber bent with the addition of a single timber pile per bent. All piling were to be driven to a minimum elevation -65.0 (NGVD1929 Datum). Pile driving records for piling driven between timber bents 33 to 41 indicate timber piling achieved capacity ranging between 17 and 65 tons (allowable using ENR formula) with a bearing elevation ranging between -49 ft to -76.5 ft. The average tip elevation and capacity between these bents were -68.5 ft and 37 tons respectively. It appears all piles achieved bearing in the medium dense portion of Unit 2 soils. All piles were driven from the bridge with no

apparent settlement of the existing bridge during pile driving. Piles were spliced with the requirement that all splices be located below elevation -2 ft.

Based on our analyses, we recommend supporting the maintenance turnout on treated timber piling. The recommended LRFD strength/extreme capacity for the timber piles is 100 kips (ultimate) assuming all piles will be driven to a minimum tip elevation of -65 ft (NADV 1988 Datum). Ultimate uplift capacity was estimated at 80 kips.

Resistance factors for bearing capacity and uplift for service, strength, and extreme limit states for driven piles in sands are shown in the table below. The resistance factors are based on using the WSDOT pile driving formula. We recommend allowing pile driving equipment with a minimum developed energy of 15 ft-kips. This would require an amendment to the Standard Specifications which currently requires hammers with a minimum developed energy of 21.5 ft-kips for air-steam hammers.

**Table 1 – Driven Pile Resistance Factors**

Limit State	Resistance Factor $\phi$		
	Skin Friction	End Bearing	Uplift
Strength	0.50	0.50	0.45
Service	1.00	1.00	N/A
Extreme	1.00	1.00	1.00

We do not recommend splicing of timber piles therefore all piles should be ordered and driven full length. We feel pile splices would adversely affect uplift capacity. Settlement of the piles under service loads will have to be checked once service loads have been determined and pile group size is estimated.

If you have any question or require further clarification of these recommendations please fill free to contact David Jenkins at PBX 5455.

T.M.A.dvj  
DVJ

cc: J. Hart, OR, WA48  
M. Morishgi, OR, 47440  
M. Hitzki, OR, 47440  
M. Sheikhizadeh, 47365

## **Jenkins, David V (MLO)**

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**From:** Jenkins, David V (MLO)  
**Sent:** Wednesday, July 21, 2004 8:53 AM  
**To:** Anderson, Mark - HQ Br; Messmer, Tony  
**Subject:** SR101 Hoquiam River/Simpson Avenue Bridge

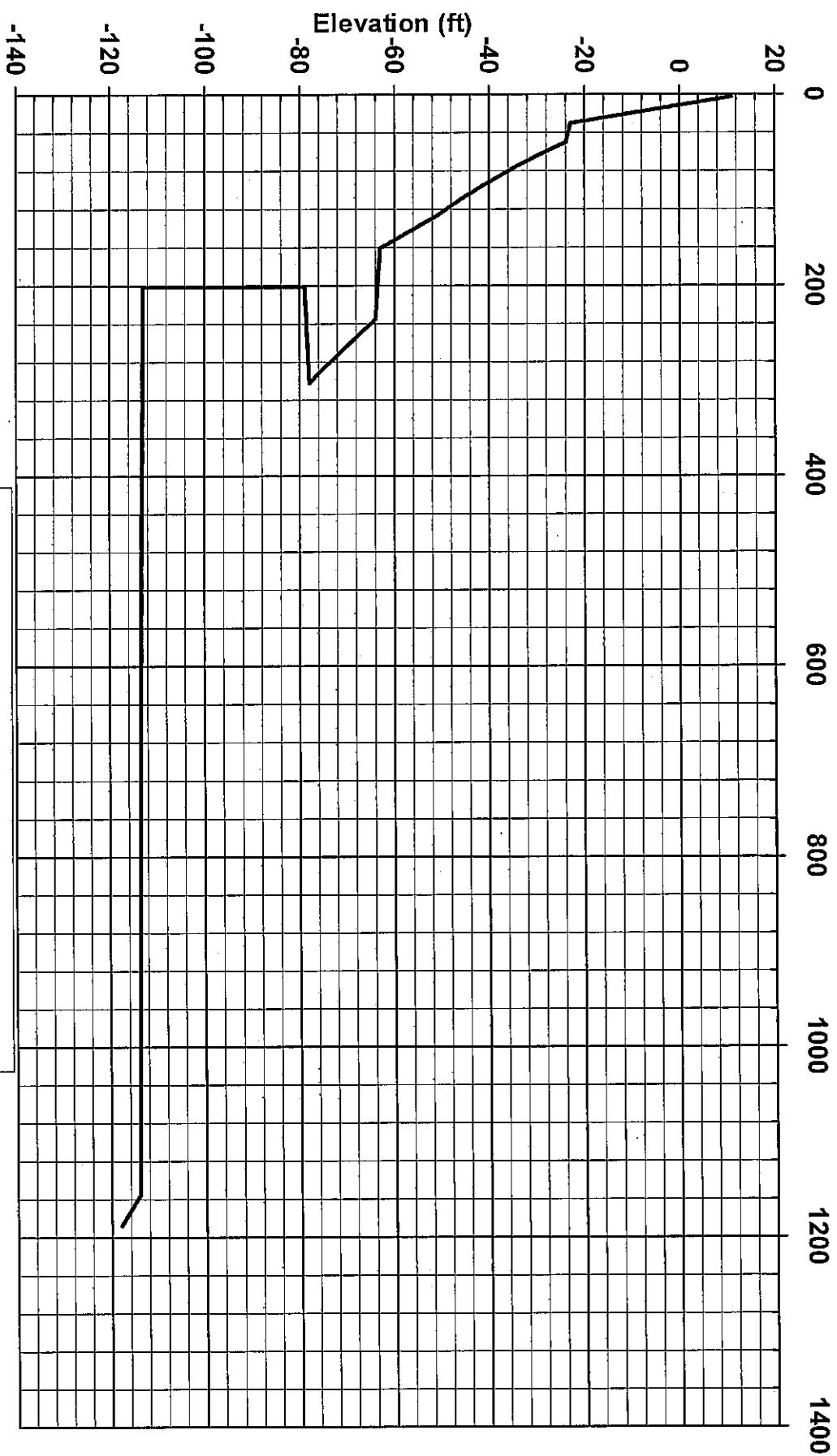
Mark/Tony

Open the attached pdf file. Contained are capacity charts for 12 inch and 14 inch cast-in-place pipe piles. You could use the piles to their full capacity which would likely result in their driving to the bedrock layer. If you design them as a low capacity pile similar to the timber pile - they may hang up in the medium dense sands located between elevation -60 and -80 ft. I would specify a minimum tip elevation of -65 ft. Please use the appropriate resistance factors which were provided in the May 12th memo.



HOQUIAM.pdf

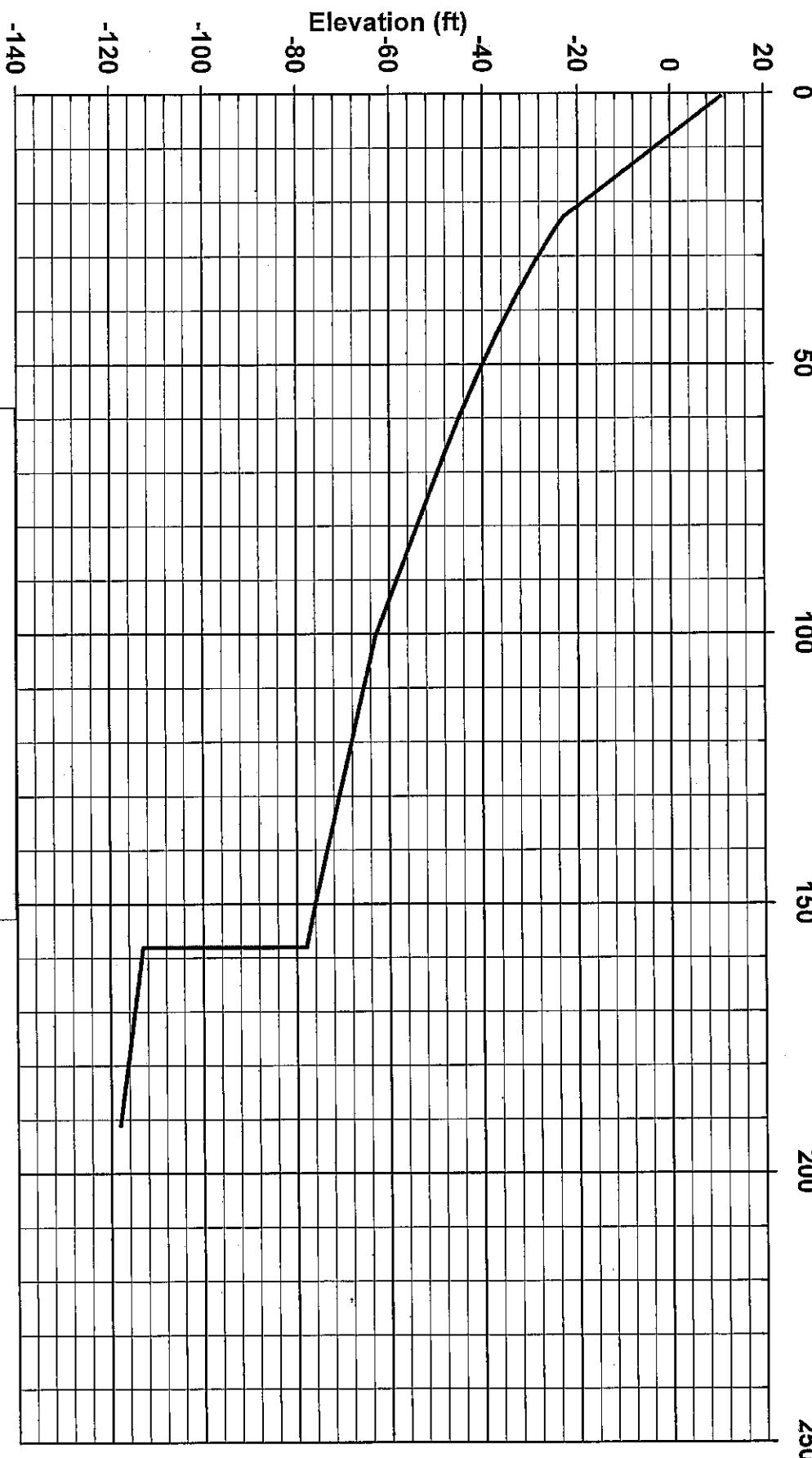
**Ultimate Bearing Capacity (kips)**  
**Strength, Service & Extreme**



— Strength, Service & Extreme

12inch pipe pile

**Strength & Extreme  
Ultimate Uplift Capacity (kips)**



— Strength & Extreme